The Structural Use of Aerated Concrete*

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Introduction

Aerated concrete is one of a group of materials described collectively as 'lightweight concrete.' The term 'lightweight concrete' is somewhat arbitrary; in the past it has been customary to consider as 'lightweight,' any concrete of dry density not exceeding 100 lb/cu. ft., but the use of reinforced units has led to concretes with a density of 110-115 or even to 120 lb/cu. ft.

The usual method of making concrete light is that of introducing air into its composition, but there are three ways by which this can be done.

(1) By using a single-size aggregate, leaving interstitial air-voids; this so-called 'no-fines' concrete has a density much lower than an 'all-in' aggregate concrete made from the same materials;

(2) by employing a porous aggregate, thereby retaining some air within the aggregate itself;

(3) by introducing air or other gas into a cement slurry in such a manner that when the cement has set, a uniform cellular product is formed. This last we call 'aerated concrete,' although outside the United Kingdom, the terms 'gas concrete,' 'pore concrete,' 'cellular concrete' and 'foamed concrete' are in common use.

Fig. 1 shows the three distinctive forms of lightweight concrete. Combinations of these are also made, for example, no-fines concrete using porous aggregate and aerated concrete containing lightweight aggregate. Fig. 2 shows a section through aerated concrete and demonstrates its cellular structure.

By suitable adjustment of its composition and mode of manufacture, a range of aerated concretes can be produced having densities between 25 lb and 90 lb/cu. ft., with a correspondingly wide range of other properties.

Aerated concrete has been described as a cement paste into which gas bubbles have been introduced, but in practice, aerated concrete always contains a considerable proportion of siliceous materials in the form of silica flour, ground blast furnace slag, pulverised-fuel ash or ground burnt shale. These materials function partly as a filler and partly as a chemical reactant with the binder, which may be Portland cement or lime.

Although aerated concrete is functionally a concrete, it is very dissimilar in its composition and it could be more appropriately regarded as an aerated mortar or aerated sand-lime product. So marked is the dissimilarity that it is perhaps unfortunate that the term concrete has been retained for these aerated products. A more descriptive name would be 'aerated silicate' or 'cellular silicate.' In contrast, the other forms of lightweight concrete are but modifications of conventional concrete.

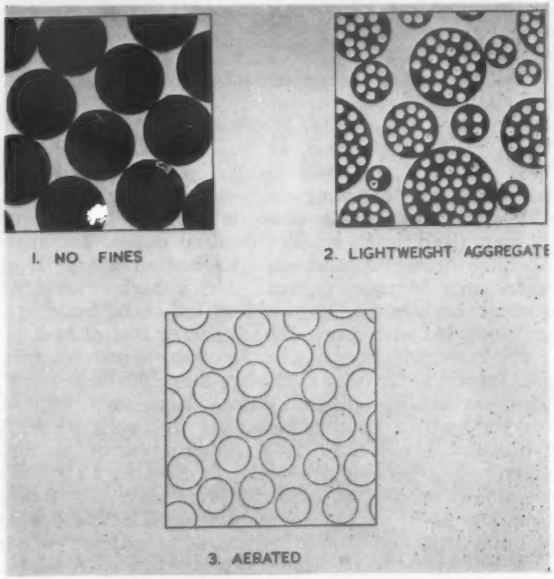

Fig. 1.—Types of lightweight concrete.

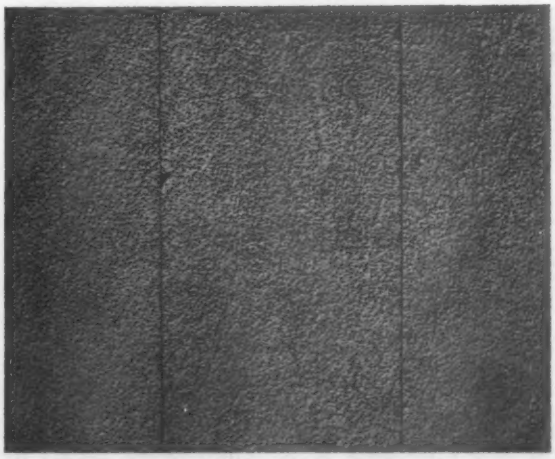

Fig. 2.—Cellular structure of aerated concrete.

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Aerated concrete is produced in two distinct forms : precast units and in-situ concrete. Precast units are usually steam-cured at high pressure ; in-situ material is necessarily air-cured. This is an important distinction, as only by high pressure steam-curing (autoclaving) is it possible to obtain a really light material with levels of strength and drying shrinkage that are acceptable for structural members. In-situ aerated concrete is used, however, for insulation roof screeds, pipe-lagging and other such purposes, for which high strength and low drying shrinkage are of secondary importance.

This paper will be confined to a consideration of autoclaved reinforced aerated concrete units in the form of wall, roof, and floor slabs, and as lintols.

The main advantages of large reinforced aerated concrete units are their low weight (thus reducing dead loads and handling loads considerably compared with ordinary precast concrete members) speed of erection (leading to smaller funding costs), and high inherent thermal insulation. This last advantage is of importance because demand for a higher level of comfort, both in dwellings and in industrial buildings, has coincided with a rapidly advancing cost of fuel.

The basic principles of aerated concrete manufacture were known as far back as the beginning of the century, but commercial production did not begin until 1929 in Sweden, with a process based on the work of Axel Eriksson. From that date the progress of aerated concrete has been impressive. In Sweden today, its use almost eclipses all other building materials in any situation where it can be employed. Its world-wide manufacture and employment in climates ranging from the Arctic to the Equator suggests a good adaptability to weather conditions. Moreover, it is now being manufactured in industrially backward countries as well as in countries with a high level of building and civil engineering technology, where it is often in competition with long-established materials of high technical merit.

Licensees of the Swedish companies operate in nearly twenty countries, and in addition, a few countries operate independently. In the United Kingdom four factories now produce autoclaved aerated concrete.

Manufacture

Aerated concrete is made by introducing air or other gas into a slurry of cement or lime containing a siliceous filler, so that when the binder sets hard there is formed a uniformly cellular structure as is shown in Fig. 2.

There are several ways in which air-cells or other voids may be formed in the slurry ; these include the following :

(1) By the formation of gas by chemical reaction within the mass during the liquid or plastic stage, in much the way that carbon dioxide is formed and used in aerating bread and other baked products.

(2) By introducing air from without, either by the use of stable foam, such as is used in fire-fighting, or by incorporating air by whipping (with the aid of an air-entraining agent) in the manner in which egg-white may be whipped up into a foam.

In the gasification method, aluminium powder is added, which reacts with lime which is either used as the cementing agent or is formed in the matrix by the setting of the cement where the latter is used as the binder. The products of the reaction are calcium aluminate and hydrogen gas. Powdered zinc may be used instead of aluminium ; in such a case calcium zincate and hydrogen are formed. Alternatively, if hydrogen peroxide and bleaching powder are used instead of metal powder, oxygen is evolved instead of hydrogen.

The only method of much practical importance today for precast autoclaved aerated concrete is the aluminium powder process. The foam methods of producing aeration are used mainly for making in-situ aerated concrete.

Autoclave-curing is virtually unavoidable in making aerated concrete of acceptable structural quality where cement is used as the binder, and essential when lime is used. With Portland cement the development of strength in the product depends primarily on the normal setting of the cement and autoclaving is used to improve the characteristics of the aerated concrete. With lime as the binder, the strength development depends entirely upon reaction between the lime and the siliceous component, i.e., where lime is used, autoclaving is necessary to make a product at all. After autoclave-curing, the product in both cases is substantially mono-calcium silicate hydrate.

In addition to the cement (or lime), sand, and aluminium powder, other ingredients may be added for a variety of reasons. For example, a wetting-agent is used to assist in the wetting of the cement and the metal powder, sugar in one form of another is sometimes added to induce solubility of the lime, saponin or other like material to stabilise the gas cells, and soluble alkali, such as caustic soda, to initiate the reaction between the metal powder and the alkaline materials present.

In practice, the sand, shale ash, slag or other siliceous material is ground in ball mills to the required degree of fineness, which is as a rule comparable with the fineness of normal Portland cement. Where pulverised-fuel ash is used, no grinding is necessary. The grinding may be by dry-mill, but wet-grinding is more often used, the fine sand produced being held as a slurry or suspension and adjusted with water to a predetermined density and maintained thus by agitation. If dry-grinding is used, the fine sand is held as a dry powder in silos, usually by this stage having been blended with the cement or lime.

In the wet process, the measured quantity of slurry, representing a known quantity of sand, is transferred to the mixer, the cement added by weight-batching, and the other chemicals in the formula, including the metal powder, introduced at this stage.

The mixing is batch-wise, one batch to each mould. The mix is transported by an agitated rail-tank to the particular mould to be charged, or the mould may be brought to the mixer. In the dry process the procedure is similar except that a dry-blended mixture of sand and lime (or cement) is added to the mixer together with the water and the other ingredients.

Identical moulds are often used whether the product is to be in the form of masonry blocks or reinforced units, indeed the reinforced slabs and the blocks are frequently made together in the same mould. The prefabricated reinforcement cages are located accurately in the mould, so that when the casting is subsequently cut into slabs, the reinforcement will be exactly where it is required in each separate slab. The batch of slurry is then run into the mould, which is only partly filled, but after about 20 minutes, by virtue of gas formation, the mixture will have become sufficiently aerated to fill the mould to over-flowing.

After a period of about six hours, the mass will have set sufficiently stiff to withstand cutting. At this point the mould is transported to the cutting

Fig. 3.—A loaded mould being delivered to the cutting machine.

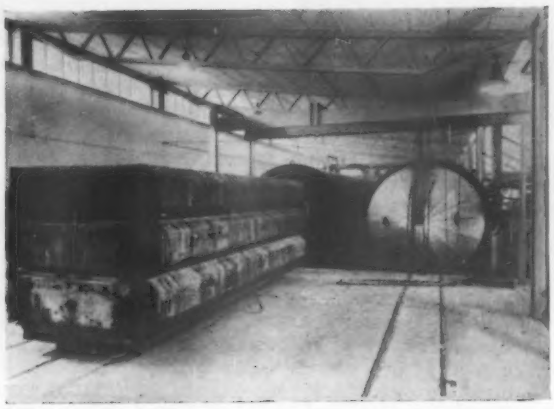

Fig. 4.—Moulds about to enter the autoclave.

machine and after trimming off the material standing proud of the mould edges, which is recovered as rework, the hinged mould-sides are dropped away to expose the casting for cutting (Fig. 3). When the parallel cuts have been made in two directions by tensioned wires (in much the way that cheese is cut) so as to produce the required blocks or slabs, the mould-sides are replaced and the whole transported to the autoclave. (Fig. 4).

The autoclaves are frequently about 10 feet in diameter and more than 80 feet in length. The aerated concrete within the moulds remains in the autoclave for 14-18 hours at about 150 lb/sq. in. gauge corresponding to a temperature of about 160°C. After this the moulds are withdrawn, the hinged sides dropped, and the contents picked up for transport to the stock-pile. The general process is illustrated in Fig. 5.

Outside of the main stream of the process outlined above, a number of essential operations take place : reinforcement rods are cut and welded into suitable cages and are dipped in the coating mixture to protect them from corrosion ; moulds are cleaned, re-oiled, and returned to their position in the casting hall for refilling.

This account is not intended to describe any particular factory or proprietary product, but to give the broad pattern of procedure within which the various brands of aerated concrete are produced. Manufacturers differ in details of manufacture, such as, the optimum times, temperatures, pressures, etc. But on one thing they do not differ : that is, the rigid control which they exercise over whatever routine they have adopted. This is necessary for the consistency of their product.

Properties

Density

Low density is probably the most characteristic feature of aerated concrete.

The figures quoted for density in the commercial literature refer to oven-dry specimens or in some cases to specimens which have reached equilibrium with normal air conditions, a difference which for most purposes is not important. They do not refer to the material as delivered from the autoclaves, which because of its moisture content may be 25 per cent heavier than the oven-dry material.

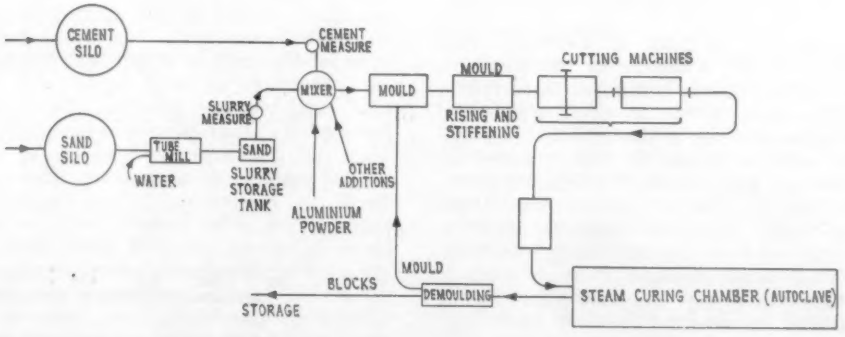

Fig 5.—Manufacture of aerated concrete. Flow sheet.

Aerated concrete is made in a wide range of densities — usually 25-50 lb/cu. ft. The lighter of these grades are used solely for insulation, but they have indifferent strength. The heavier grades, 40-50 lb. per cu. ft., have lower insulation value and have higher strength. The intermediate density-grades show a gradation of strength with a corresponding gradation in the thermal conductivity. Even the heavier grades of autoclaved aerated concretes, however, have higher levels of insulation than other structural materials.

The low density of aerated concrete leads to a substantial reduction of the dead-load of buildings and of the loads which have to be transported and handled. Structural reinforced aerated concrete has been used at densities between 30 and 40 lb/cu. ft., that is, at one-fifth to one-quarter of that of conventional concrete, and although larger sections may sometimes be necessary with aerated concrete in order to obtain the necessary rigidity or strength, the saving in weight is always considerable.

Compressive Strength

The assessment of declared strength values from different sources is rendered difficult because of differences in testing methods and in the shape and size of specimens. The effect of such differences is not trifling: size and shape can influence the results by 30 per cent; specimens tested in the saturated condition may reach only 80 per cent of the strength of the same concrete when tested dry. Consideration has been given therefore only to values produced by laboratories whose testing methods are known.

From many tests on commercial products, it is possible to give the average compressive strengths which might be expected from autoclaved aerated concrete within the density range 25-50 lb/cu. ft., when tested by the British Standard method for blocks, that is, the mean value for 12 full blocks, mortar capped to parallel faces, loaded in the surface dry, saturated condition.

Fig. 6 shows that material of 35 lb/cu. ft. and over can be expected to have a compressive strength of at least 400 lb/sq. in., which is accepted for lightweight aggregate building blocks in BS.2028. There may be some considerable departure in strength, however, where a particular brand of aerated concrete employs a composition formula substantially different from that outlined in the preceding section on manufacture.

Fig. 6 also shows that within the range of densities examined on commercial products there appears to be a straight line relationship between strength and density.

Some studies have been made in Germany and Sweden as well as in this country, on the effect of age and storage conditions. It is difficult to correlate the results but, in general, storage in water or in damp conditions leads to an increase in strength with ageing. However, if the water in which the test specimen is immersed is in motion, leaching of the residual binding material might take place with a consequent loss of strength. This might offset the potential gain in strength to be expected from the continuing hydration which can take place in wet storage. It seems unlikely that much change, either way, will take place due to ageing, in the absence of moisture.

Tests on cylinders under prolonged loading in controlled temperature and humidity conditions[1] indicate that the compressive strength is likely to be

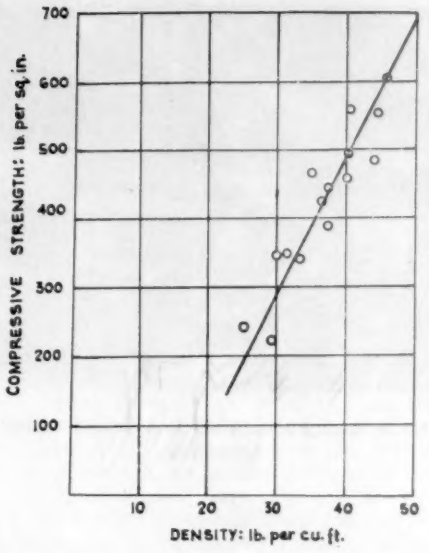

Fig. 6.—Relationship between compressive strength and density for autoclaved aerated concrete.

reduced by about 20 per cent as compared with that under short duration loading. A similar reduction has been observed with ordinary dense aggregate concrete.

Modulus of Elasticity

The modulus of elasticity of aerated concrete is low when compared with conventional concrete. Figures are available from a number of laboratories, and these are given in Table 1 and in Fig. 7.

The modulus of elasticity of ordinary concrete is normally between 3×10^6 and 5×10^6 lb/sq. in.

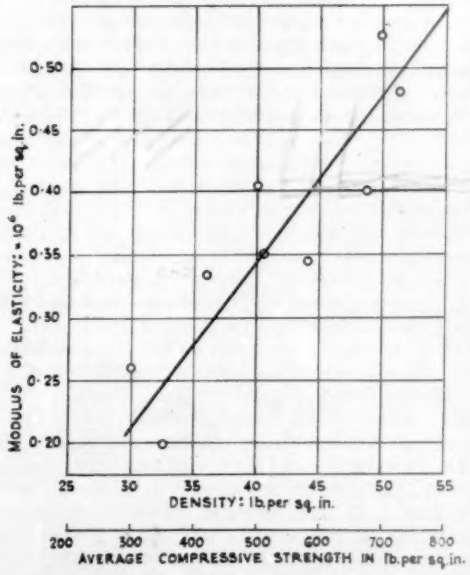

Fig. 7.—Relationship between compressive strength and modulus of elasticity for autoclaved aerated concrete.

Table 1,—Modulus of elasticity of autoclaved aerated concrete

Dry Density lb. per cu. ft.	Modulus of elasticity in lb. per sq. in × 10^6, obtained by			
	Building Research Station	Vinberg[3]	Manufacturer's Laboratory (A)	Manufacturer's Laboratory (B)
30	—	—	0·235	—
31	0·20	—	—	0·20
32·4	—	0·195	—	—
37	—	—	—	0·31
40	—	—	0·380	—
40·5	—	0·324	—	—
44	0·32	—	—	0·32
47·4	—	0·376	—	—
50	—	—	0·520	—
51·1	—	0·452	—	—

Creep

In addition to the elastic deformation, aerated concrete, like other cement products, undergoes plastic deformation or creep under sustained loads. Sustained loading tests on reinforced slabs and prisms of 31 lb/cu. ft. aerated concrete were reported by Båve[3] and Schäffler[4], and compared with similar tests on dense concrete. Tests have also been made at the Building Research Station.

The indications are that creep effects at working loads will not be greater with aerated concrete than those obtained with ordinary dense concrete.

Drying Shrinkage

All cement products are subject to small changes in volume, accompanying changes in moisture conditions. Though small in actual dimensions, such changes are important in terms of the functional behaviour of the concrete or mortar. Shrinkage was formerly attributed solely to the cement, but recent work has shown that a few natural aggregates have marked drying shrinkage and so contribute to the overall shrinkage of the concrete in which they are used. Neat cement has a very high drying shrinkage however and in general a hard rigid aggregate, such as is used in dense concrete, restrains to some degree the shrinkage of the product. Lightweight aggregate, having a cellular structure, does not restrain shrinkage to the same extent, and lightweight aggregate concretes normally show a somewhat higher drying shrinkage than dense concrete.

Aerated concrete usually has no aggregate other than a fine filler and this in any case is in much smaller proportion than is common in other concrete. Consequently aerated concrete produced by ordinary air-curing has a very high drying shrinkage. However, when the product is cured in high-pressure steam, fundamental changes take place in the mineral constitution which may reduce the shrinkage to one-quarter or even one-fifth of what it would have been with air-curing. Midgley[5] has shown that in autoclaved aerated concrete which he has examined, the main cementing agent was well-crystallised tobermorite, a mono-calcium silicate hydrate, giving on an X-ray diffraction pattern, a well-defined reflection at about 11·4A. On the other hand, air-cured aerated concrete from the same manufacturing batches showed poorly crystallised tobermorite or tobermorite-gel, the X-ray diffraction showing it to be a highly disorganised state. This work neatly demonstrates for aerated concrete a belief which has been held for many years in respect of other concretes, that a gel-form of set-cement is associated with high shrinkage and that this gel-form is changed to a micro-crystalline form by the conditions of high-pressure steam-curing.

It might be said therefore that an efficiently autoclaved aerated concrete is a different material from the corresponding air-cured product made from the same raw materials.

Resistance to Rain-Penetration

Practical experience in Sweden indicates that the resistance to rain penetration of aerated concrete is satisfactory except under the most severe conditions, when many other materials would likewise fail. In such cases of severe exposure, it has been found desirable to clad the exposed walls with thin asbestos-cement tiles. Even though complete penetration of liquid water may not occur, it is undesirable that the aerated concrete should become damp, as the thermal insulation is adversely affected by moisture.

Many workers have reported on the moisture content of aerated concrete in equilibrium with normal air humidity, and although the values differ somewhat, the mean value appears to be about 5 per cent by weight. However, surveys made in Sweden have shown that moisture contents up to 40 per cent or more can exist in very exposed positions.

In this country aerated concrete blocks rendered externally, and reinforced wall-panels sprayed with a protective paint have been subjected to an artificial rain test for periods exceeding six hours, without showing signs of water staining on the back-face. Such a result is held to indicate adequate resistance to all but the most exposed conditions.

Thermal Insulation

In recent years more attention has been given than formerly to heat-losses from buildings, to reduce fuel consumption while improving comfort conditions both in industrial and domestic buildings. In the domestic sector, recommendations have been made from time to time on the desirable standard of thermal insulation, and more recently legislation has been enacted to control the standard of insulation of industrial buildings.

One of the principal features of aerated concrete is its low thermal conductivity, it being roughly proportional to the density of the material, as shown in Fig. 8.

The insulating character of building materials is expressed either as the ' k-value ' or the ' U-value.'

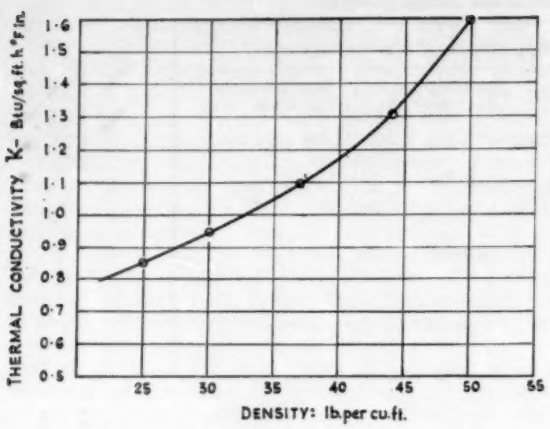

Fig. 8.—Relationship between thermal conductivity and density for autoclaved aerated concrete.

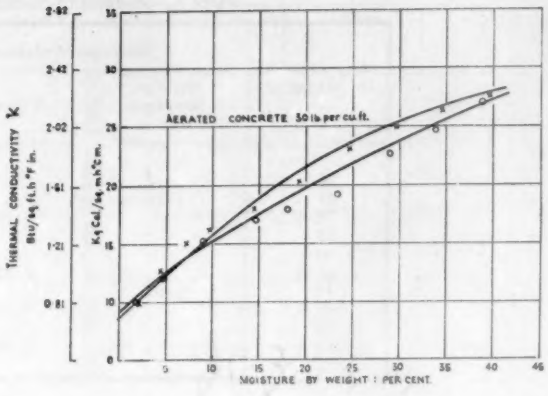

Fig. 9.—Effect of moisture content on thermal conductivity.

Thermal Conductivity or k-value is the quantity of heat, expressed in British thermal units, transmitted in one hour through one square foot of homogeneous material, one inch in thickness, for a temperature difference of 1° Fahrenheit, from face to face.

Thermal Transmittance or U-value of a wall (which is not necessarily homogeneous) is the quantity of heat, expressed in British thermal units, transmitted in one hour, through one square foot of the wall when the temperature on the two sides of the wall differs by 1° Fahrenheit.

Whilst the k-value is a property of a homogeneous material, the U-value refers to heat transfer through a specific wall system, such as brick or brick and concrete, plastered or plastered and rendered, with or without single or multiple cavities, and of some specific thickness. In comparing the insulating value of various concretes, it is convenient to use the thermal conductivity or k-value as a basis and Table 2 shows typical k-values for several densities of aerated concrete, compared with conventional heavy concrete.

Moisture substantially affects the thermal conductivity of materials. This was shown many years ago by Cammerer[6] and more recently by van Gunst and van Zuilen[7]. Still more recently Granholm[8] has given in tabular form the relationship between moisture and thermal conductivity in aerated concrete of density 30 lb. per cu. ft. (Table 3). This shows the deterioration in insulation value which accompanies increase of moisture content.

Jonell[9] likewise studied the effect of moisture on aerated concrete at different temperatures. Typical curves from his work are given in Fig. 9. In these cases the material had a density of 30 lb/cu. ft. and the temperatures were 2°C and 22°C.

Table 2.—Thermal Conductivity Values

Material	k-value B.t.u/sq. ft. h°F in.
Dense Concrete	8 to 12
Aerated Concrete 25 lb/cu. ft.	0·85
30 ,,	0·95
37 ,,	1·10
44 ,,	1·30
50 ,,	1·55

Some Structural Characteristics

General

Aerated concrete was first used in load bearing members in the form of masonry walls.

Having successfully overcome the difficulties—in themselves formidable—of developing large-scale processes of manufacture, Swedish engineers then successfully tackled the problem of making autoclaved aerated concrete flexural members spanning up to 20 ft., capable of carrying heavy design loads, without excessive deflections and without corrosion of the reinforcement.

In this development of reinforced aerated concrete units a number of important problems arise which include the protection of the embedded steel against corrosion ; the maintenance of adequate bond strength ; the choice of an optimum density and compressive strength for economy and lightness ; the effects of shrinkage and creep ; the occurrence of cracking and failure in aerated concrete beams ; the development of a logical and safe design theory ; the transverse stiffness of slabs made up of aerated concrete members

Answers to some of these important problems have already been obtained. Research is at present in progress on others. The need for urgency is enhanced by the fact that large scale manufacture of aerated concrete building elements is taking place throughout the world, including the United Kingdom.

Protection Against Corrosion

Causes of corrosion.—For some twenty years reinforced aerated concrete elements were made in Sweden without special protection for embedded bars. In roofs and floors used for ordinary domestic buildings little serious damage due to corrosion of the steel is said to have occurred.

Aerated concrete, owing to its porosity, is however inherently less resistant to moisture penetration than dense concrete and takes up carbon dioxide from the air more readily. Normally steel embedded in concrete enjoys a certain degree of protection from corrosion because of the alkaline nature of concrete, but if the concrete is carbonated, this immunity is lost. Steel in aerated concrete is therefore vulnerable both on account of the access of moisture and the ease of carbonation. In aerated concrete, moreover, most free alkalis are combined in the autoclave and bare

Table 3.—Moisture effect on conductivity (Granholm's results)

Moisture per cent by weight	k-value of aerated concrete (density = 30 lb/cu. ft.)
0	0·8
5	1·0
10	1·2
15	1·4
20	1·6
25	1·8
30	2·0
35	2·2

embedded steel bars tend to become vulnerable to corrosion if the concrete is exposed to humid atmospheric conditions, e.g. where building elements are exposed to the weather or where condensation occurs in unventilated roof spaces, or with certain industrial processes. Protection against corrosion of the reinforcement is therefore necessary, particularly where the atmospheric conditions are not only humid but also laden with corrosive impurities as in most industrial areas of the United Kingdom.

Methods of protection.—For ordinary reinforced concrete, the reinforcement must be surrounded by a sufficiently thick cover of well compacted concrete to prevent penetration and diffusion of moisture and of any corrosive substances it may carry. In aerated concrete the effect of the thickness of concrete cover on the liability of embedded reinforcement to corrode is of subordinate importance, reliance being placed primarily on the direct protection of the steel by repeated immersion in protective material. After drying, the reinforcement is left covered with an impermeable coating. The material used must fulfil stringent requirements with regard to resistance to heat, brittleness, ageing, mechanical strength and chemical inertness. Paints are unsatisfactory because in general they cannot resist the high temperatures and moisture in the autoclave. Phosphate treatments were found to be wanting because of the brittleness of the surface layer. Bituminous solutions of various kinds are usually satisfactory but in most cases lead to unreliable bond properties. Cement slurry coating by itself was not enough to provide protection over a long period but when mixed with some other materials and binders, e.g. rubber-latex, the protection provided was adequate and the reduction in bond strength was in general less than with bituminous mixes.

Exposure tests.—Tests have been made by the Building Research Station at an exposure site provided by the London County Council at Beckton with a large number of specimens each containing several reinforcing bars with different depths of cover and different types of protective coating (Fig. 10). Exposure conditions were severe, such as might occur in some industrial areas of the United Kingdom. Specimens were removed at intervals from the site; they were broken up and rusting of the bars was first visually assessed. The amount of steel lost in each specimen due to rusting was obtained by weighing the clean steel bars before embedment and again after extraction, after the bars had been freed of rust. Four conditions were examined, namely, bars provided with no coating; bars with a proprietary phosphate treatment, followed by a paint sealer; bars coated with cement slurry and bars with a bitumen coating.

The results obtained are shown in Fig. 11. Average values for the ratio $R = \frac{w}{w_1}$ are shown in relation to the time of exposure. In this ratio, w denotes the weight of steel lost due to corrosion for $\frac{1}{2}$ in. diameter bars subjected to the stated protective treatments, when embedded with a concrete cover of $1\frac{1}{2}$ in. and w_1 denotes the weight of steel lost due to rusting for bare bars embedded with the same cover in the same specimen.

For bare bars, i.e. bars which had not been coated, corrosion in the autoclave was followed by rapid increase of rusting due to exposure, and consequent cracking up of the concrete cover after two or three months. Their average rate of corrosion is shown in Fig. 12.

Where the bars had been subjected to a phosphate treatment and then painted, the autoclaving destroyed the paint. Subsequent exposure led to rapidly increasing corrosion; after two years it reached about one half of that for the bare bars (Fig. 11).

For bars covered with cement slurry, loss of steel due to corrosion continued, but in relation to the behaviour of unprotected bars the ratio R remained constant.

For bitumen covered bars the loss of steel was negligible.

Flexural members similarly exposed in which the reinforcement was protected with proprietary cement-rubber latex coating were examined at intervals; after two years' exposure no signs of rust were seen.

The results of the exposure tests at Beckton under severe conditions of moisture penetration and atmospheric pollution indicate that it would be unwise to use unprotected bars for reinforced aerated concrete members. Although it seems unlikely that they would be permanently under such severe conditions they may be exposed to such conditions for varying periods during construction. Nor is it possible to ensure throughout the life of a structure that it will not have to resist harmful influences, such as high humidity and air pollution, arising from certain industrial processes. Of the protective treatments used, the bituminous and the cement-latex proprietary coatings both seem to provide long-term protection.

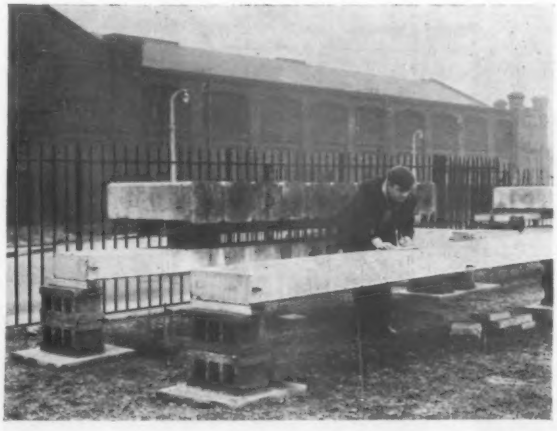

Fig. 10.—Aerated concrete beams on the exposure site at Beckton.

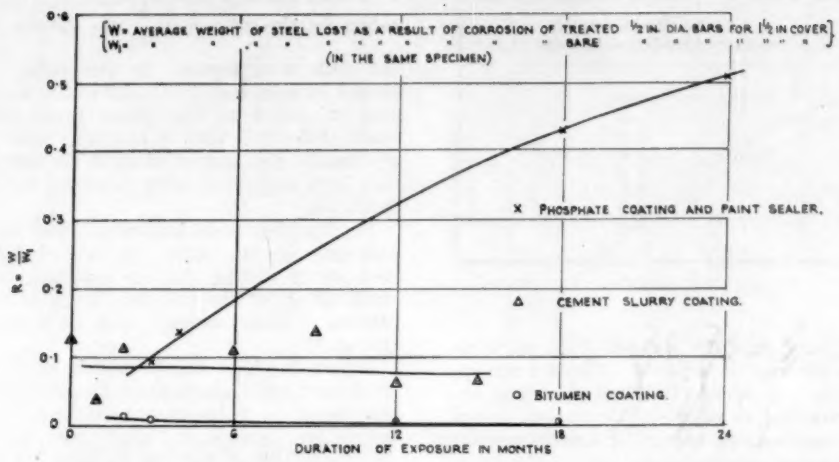

Fig. 11.—Effect of duration of exposure on average corrosion of reinforcement.

Certain materials used abroad for the manufacture of aerated concrete are inherently corrosive. Protection against attack on the steel is then doubly necessary. Fortunately, none of the materials used at present in the United Kingdom for the manufacture of aerated concrete is inherently corrosive.

Durability

In addition to protection against corrosion of the reinforcement, the durability of aerated concrete itself is of obvious importance.

Exposure of beams to the weather at Beckton in an atmosphere rich in sulphur led to the appearance of efflorescence and after a time to the growth of algae. Cracking parallel to embedded reinforcement occurred on exposed members after about three months exposure, both near the tensile and compression reinforcement. These cracks were not in general due to rusting of the reinforcement and may have occurred as a result of manufacturing faults accentuated by weakening of the material due to exposure.

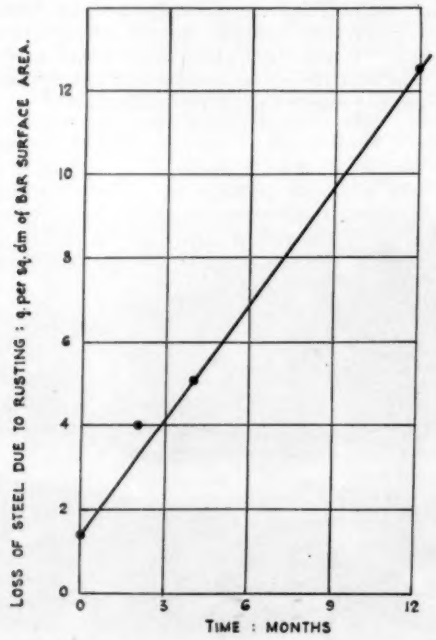

Fig. 12.—Effect of duration of exposure on the average loss of steel due to corrosion in bare ½ in. diameter bars with 1½ in. cover in aerated concrete.

Exposed aerated concrete floor and roof members were loaded to about one-half the ultimate load for identical members under short-term loading in the laboratory, i.e., subjected to a load equal to about double the design loading. The permanent deflection of one of the members is shown in Fig. 13, together with the deflection of a similar beam loaded in the laboratory. The member tested on the exposure site failed after about nine months exposure when its additional permanent deflection reached about 75 per cent of the initial elastic deflection.

Failure was probably caused by shear fracture. A series of tests was made to evaluate the effect of different factors on this reduction of the strength to about half of that obtained in the laboratory for short-duration loading tests. The following conclusions were reached :

(a) For short duration loading, saturation with moisture reduced the strength by about 13 per cent compared with that for storage indoors.

(b) Prolonged loading at about one-half of the ultimate short-duration load for a period of about nine months had, in itself, no appreciable effect on the strength.

(c) Exposure to a polluted atmosphere for nine months reduced the strength by about 40 per cent.

The effects of sulphur were shown to be important by observations made on the aerated concrete roof of a steel works in Sweden[11] where the internal atmosphere contained sulphur and the soffit was not protected. About two years after installation the slabs started to deform excessively, and in due course had to be replaced. An extensive field survey of the roof was carried out and the cause of this behaviour was found to be related to the absorption of sulphur

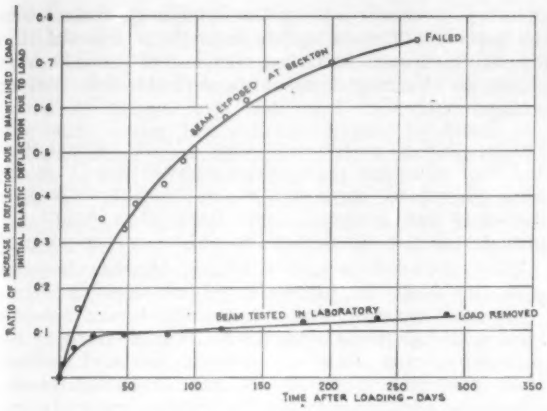

Fig. 13.—Deflection of slabs under sustained loading.

gases in the lower part of the slabs, leading to expansion of the concrete and excessive deflections. A further investigation is being carried out with slabs protected with various surface treatments.

Although the sulphur content of the air at Beckton is unlikely to be as high as in a steel works, it seems that combined with the presence of moisture it led not only to an increase of the deflection under load, but also to a considerable reduction in strength.

Bond and Anchorage

General.—Reinforced aerated concrete must rely for stiffness and strength on effective bond between the concrete and its reinforcement. The effectiveness of the bond also greatly affects cracking in beams and slabs.

Pull-out tests do not provide wholly reliable evidence for a quantitative index of bond strength with different types of concrete or bar, either for dense or aerated concrete. Nevertheless, they can provide a qualitative assessment of the effect of various factors on the bond strength.

The effect of the diameter and position of the bars.—For aerated concrete as for ordinary well-graded gravel concrete, bond strength in general increases with decreasing bar diameter, but for bars held horizontally during casting, there is a levelling off or even a reduction of average bond strength for bar diameters less than $\frac{1}{2}$ in. (Fig. 14), particularly for bars placed in the upper part of the mould.

This is due to the manufacturing process. The bars in the lower part of the mould are covered with the liquid mix, and generally remain fully encased. The bars above are gradually being reached by the concrete swelling upwards due to aeration and it rises past them leaving cavities of varying size and shape next to the top of the bars (' Shadow effect '). The bond strength of the top bars is therefore usually lower than that of the bottom bars. For small diameter bars the proportion of total circumference which is affected is greater than for bars of larger diameter.

The effect of autoclaving.—An initial layer of rust is always created on the surface of embedded bare bars, due to exposure to steam in the autoclave at a temperature of about 160°C. The surface texture of the bars becomes rougher and bond is improved. The ratio of bond strength to compressive strength for unprotected round steel bars is higher for aerated concrete than for other types of concrete.

The effect of bituminous coating.—None of the impervious coatings used to protect the reinforcement against corrosion is without effect on the bond strength.

For bituminous coatings, frictional resistance between steel and concrete is reduced considerably, and adhesion may break down on either side of the coating, particularly where the bars are smooth and free of rust initially. The bond strength obtained from pull-out tests by different research workers varies considerably, ranging from nil to about 150 lb/sq. in., for compressive strengths ranging between 500 and 750 lb/sq. in.[4]

It has been found that, when dipping slightly rusted reinforcement in certain types of bitumen, droplets tended to form along the bars, creating a corrugated surface when dry. After hardening, such a surface has considerable bond resistance.

Under long duration static loading the bond strength of bitumen coated bars has been reduced by about 20 per cent of the bond strength under short duration loading.[4]

The effect of cement coating.—The bond strength of bars dipped in a proprietary cement grout containing rubber was found to be higher than that of smooth, bitumen covered bars, mainly as a result of corrugations formed on the bar surface by hardened droplets.

Pull-out tests made at the Building Research Station on specimens cut from aerated concrete slabs having a dry compressive strength of about 400 lb/sq. in. showed that the bond strength based on the circumference of the bar varied from about 150 lb./sq. in. to about 220 lb/sq. in. Adhesion between the steel and the cement coating remained unaffected, failure being the result of a breakdown of the concrete in shear, along the bar. For smaller diameter bars the bond strength was somewhat less. The position of the bar in the mould had no appreciable effect on bond strength since the ' shadow effect ' was less apparent.

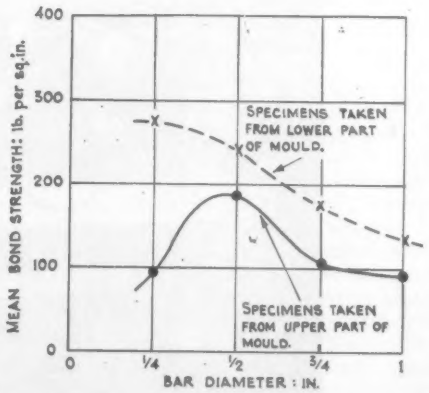

Fig. 14.—Effect of diameter and position on the mean bond strength for bare bright steel bars embedded horizontally in autoclaved aerated concrete pull-out specimens.

The effect of saturation.—The effect of saturating the specimens with moisture caused a reduction in bond strength of about 25 per cent.

The effect of crushing strength.—Bond strength increases with compressive strength but the relationship does not appear to be linear. It seems that varying surface characteristics have a more important effect than concrete strength.

The effect of deformed bars.—Adhesion and friction between steel and concrete are not in general adequate to resist the internal forces in aerated concrete flexural members. After slip has taken place, these forces must be resisted by mechanical anchorage provided by the corrugated surface texture of some coatings along the reinforcing bars. Where a cement-coating was used with ribbed bars, the space between the ribs became filled with the protective material, and bond failure took place due to shear in the concrete along the bar, as with ordinary round bars. With a thin bituminous coating on the other hand, crushing of the concrete took place due to high anchorage stresses at the interfaces between steel ribs and the aerated concrete, at relatively low loads[12].

The effect of anchorages.—Bond can be assisted by anchorage elements at the ends of the bars or intermediately, at intervals along the bars. End hooks increased the failing load in some cases by as much as 30 per cent, compared with similar specimens containing straight bars, but the results showed considerable scatter[4].

Tests on welded end-anchorage plates and angle-section shear connectors showed that these were effective in concentrating the anchorage forces near the supports of slabs and beams. It was found that for pull-out specimens failure occurred at the anchorage plates, at a bearing stress of about double the crushing strength.

The effect of welded cross bars.—With normal floor and roof members the reinforcement is usually in the form of welded cages containing transverse bars at the ends and intermediately along their length, to provide mechanical anchorage and to act as stirrups.

The effect of such anchorage elements becomes generally smaller with increasing bond strength. Thus, cross members were more effective for bitumen-coated bars than for those coated with cement-slurry. For pull-out specimens the slip between bar and concrete was appreciably reduced by increasing the number of cross bars. For flexural members, however, the number of cross bars had little effect on maximum deflection and crack formation.

Where the concrete cover was less than $\frac{1}{2}$ in., the bond and anchorage strengths were reduced considerably.

The maximum force which cross members are capable of resisting can be determined with empirical formulae[13].

In addition to horizontal anchorage bars, both vertical and inclined stirrups connecting tension and compression reinforcement have been used to resist internal shear forces and to increase bond strength. Welded stirrups inclined at an angle of 45° were found to be more effective in preventing slip than other types of cross member.

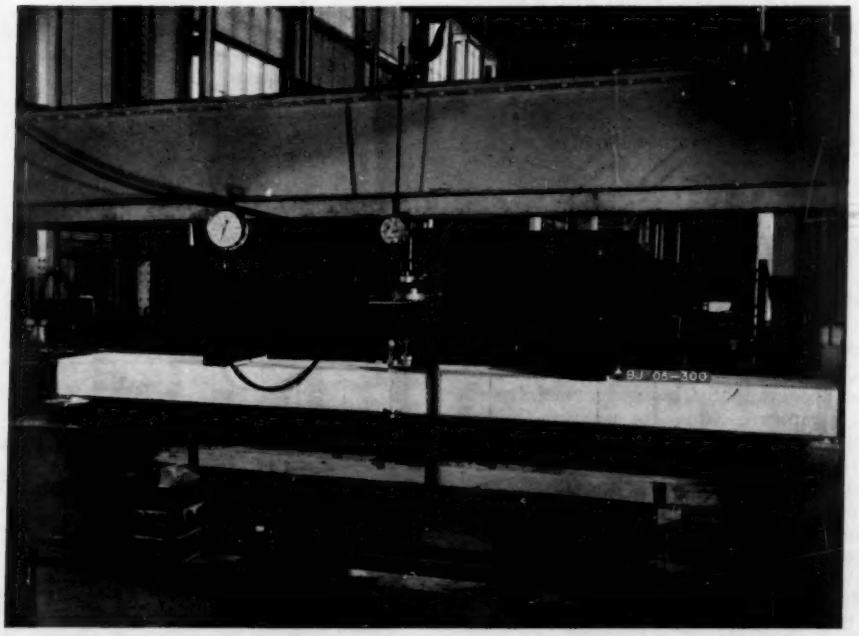

Fig. 15.—Aerated concrete beam being tested in the laboratory.

Load Factors Against Cracking and Failure

The design of aerated concrete beams is largely governed by the low strength and low modulus of elasticity of aerated concrete, but a generally accepted method of design does not exist. Manufacturers' preliminary design methods are apparently based on the elastic theory, maximum deflections under working loads being limited to 1/400th of the span.

The behaviour of such beams under short duration loading tests is known. Tests made at the Building Research Station on some beams of Swedish make provide typical results. The roof and floor members tested were of two densities, 31 and 44 lb/cu. ft.; they were designed for permissible imposed loads ranging from 23 to 82 lb/sq.ft. They were 5 in. to 8 in. thick. (Fig. 15). At design imposed loads the maximum deflections did not exceed 1/360th of the span and no cracking was observed. Failure occurred at about four times the total design load. Primary failure occurred in about half the tests as a result of yield of the tensile steel, in the remainder as a result of crushing of the concrete. Buckling of the compression reinforcement and shear failure in the concrete appeared to be secondary phenomena.

At working load the steel stresses reached about 12,000 lb/sq. in., i.e. about one-fifth of the yield stress, the maximum compressive stress in the concrete being about one-third of the cube strength, which was about 400 and 700 lb/sq. in. for the two densities used.

Cracking of the concrete became visible only after considerable deformation had taken place, corresponding to a steel strain of 800 to 1,200 $\times$ 10^{-6}, at about 65 to 80 per cent of the ultimate load. The load-deflection relationship indicated however that cracking may have occurred earlier, at about three-quarters of the visible cracking load, since observation of cracks on the porous surface is difficult. Nevertheless, the load factor against cracking was high. It seems that this property is associated only with those types of aerated concrete in which the grain size of the aggregates is of the same order as that of the binder, and is attributed to several factors :

(a) The tensile strength of aerated concrete, in common with other weak concretes, tends to be high in relation to its crushing strength.

(b) Cracking may be inhibited by thermal pre-stressing due to differential contraction of concrete and steel when cooling down after extraction from the autoclave.

(c) Aerated concrete has a high extensibility at incipient failure.

Some Design Considerations

Present Codes of Practice

The use of reinforced aerated concrete is not at present regulated by a Standard Code of Practice in the United Kingdom. In Germany, a definitive Code of Practice was issued in 1958 (DIN 4223). A Draft Code has been issued recently in Finland, and in Sweden and Norway codes are being prepared now.

These definitive or proposed Specifications stand on their own, not being dependent on the Codes of Practice relating to ordinary reinforced concrete.

Apart from laying down rules for control tests and methods of construction, the German Code of Practice for aerated concrete contains detailed regulations for the design of flexural members by means of a load factor method. The Scandinavian authorities on the other hand are of the opinion that the available test results do not justify the adoption of precise design formulae. They consider moreover that, in Scandinavia, design engineers are not in fact called upon to make detailed design calculations, since these are made by the manufacturer. The forthcoming Scandinavian Codes of Practice will therefore not be based on a theoretical design method, but on acceptance tests to destruction.

Design of Flexural Members

Design bases.—Where the design calculations are replaced by standard acceptance tests for mass-produced elements, as in Scandinavia, the tests must take account of conditions at the site, and the methods of manufacture must guarantee consistent quality of the product. Precedents for such acceptance tests already exist in this country in the Code of Practice for prestressed concrete CP.115-1959 and that for walls CP.111-1948.

Where design calculations are used however either as a basic method of design or—where a system of acceptance tests is used—as a preliminary step for manufacture, they may be based either on the elastic theory or on ultimate strength, as in the German Code. The elastic theory is used by the Swedish manufacturers. With it, the steel and concrete stresses might be reasonably calculated with a modular ratio of 150 to 220, corresponding to the low modulus of elasticity of aerated concrete.

Whichever method is used, account must be taken of the reduction in strength and increase of deflections which may result from prolonged exposure to humid conditions or prolonged loading. It is desirable to restrict deflections by adopting maximum permissible span/depth ratios for various types of structure.

Bond strength.—The bond stresses in reinforced aerated concrete structures depend largely on the type of coating used to protect the bars against corrosion and also on the anchorages used to assist bond. For some types of coating some manufacturers assume that a part of the internal forces are transmitted directly by bond while the rest is transmitted by welded cross bars, end anchorage plates or end hooks. To take account of manufacturing faults, such as 'shadow effect,' inadequate welding or insufficient surface corrugation of the coating, it may be preferable however to rule that the entire force in the bars should be resisted by anchorage forces for all types of surface treatment.

Shear strength.—A number of the specimen beams tested at the Building Research Station failed in shear, particularly after being subjected to prolonged loading under exposed conditions. Where aerated concrete beams are to be used as separate members (e.g. lintols), it would therefore seem reasonable that the internal shear forces should be wholly resisted by embedded reinforcement of suitable design. Where the beams are joined together however with the intent that they should form a monolithic slab structure capable of resisting transverse bending moments, such a requirement would appear to be unnecessarily severe. It may be assumed then for design that the concrete can resist shear stresses, but the number of beams joined together must be such as to form a slab with a width of at least half the span.

Deflections.—In view of the high load factor against cracking, the deflections might be calculated on the assumption that the concrete section is uncracked.

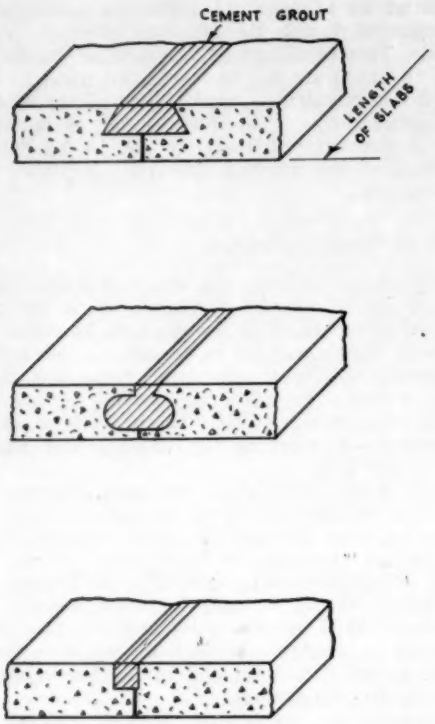

Fig. 16.—Types of edge grooving along length of reinforced aerated concrete foor and roof slabs.

Wall, Floor and Roof Slabs

Since most floors and roofs made with reinforced aerated concrete units are likely to be in the form of slabs, the efficiency of the methods used for joining individual members along their length will probably govern the ultimate loadbearing capacity and stiffness of the structure. To ensure interaction between adjacent beams it is usual to form a groove at the top, along the whole length of each beam and fill it with cement mortar. The continuous groove is formed by leaving a shaped step on one or both sides of each beam (Fig. 16). Wedge shaped grooves appear to be preferable. To improve continuity and stiffness, it is usual to insert an additional steel bar in each groove, to be encased in cement mortar.

Due to shrinkage and cracking of the joint, complete and lasting interaction between precast members is difficult to achieve. In many cases an additional cement mortar screed is spread on top of the beams, to improve lateral interaction and the stiffness and strength of the slab, as well as sound insulation.

The stiffening effect of the walls, floors and roof slabs in transmitting wind forces as well as vertical loads is not generally taken into account in multi-storey framed structures. In frameless loadbearing cross-wall structures, however, the walls, floors and roof panels are deemed to act as vertical and horizontal membranes capable of transmitting racking forces and bending moments. In the absence of test results and experience, aerated concrete units are not considered to be suitable at present for these latter structures, if more than two storeys in height.

Workmanship

Aerated concrete is relatively soft and friable. Reinforced members made with it are therefore easily damaged during manufacture, transport or erection and unsound design of the support conditions of individual beams can lead to damage followed by rapid deterioration. For successful production, accurate control of manufacturing processes is needed even without specific regulations. An equally strict control and supervision of the processes on the site and during erection however is necessary and more difficult to achieve.

Elements should not be allowed to stand exposed to the weather on sites, nor should they be allowed to become wet during erection. Support conditions should be so designed that beams are adequately bedded at their supports. Breaking of corners leading to exposure of the reinforcement can occur and repairs are rarely entirely satisfactory.

Methods of Construction

General

Apart from building blocks for masonry construction, the principal structural forms in which aerated concrete building elements are used in the United Kingdom and abroad are reinforced floor, roof and wall units and panels (Table 4) and also large wall blocks and staves, usually not reinforced. Most of these are made as standard articles capable of being fitted into various plans or methods of construction.

Single purpose units such as large panels of storey height and dwelling unit widths for multi-storey construction are made in large numbers in the U.S.S.R. with very large autoclaves, and are reported to have been satisfactory[14]. The large investment necessary for the manufacture of these large single purpose elements seems hardly justified however in general in Western Europe, due to our tendency for greater variability in plans and appearance and a greater diffusion of the building industry.

Most of the practical experience on reinforced aerated concrete construction has been obtained abroad, but the materials used and the methods of manufacture do not in fact vary greatly in different countries. The structural properties of the resulting manufactured articles also tend to be similar.

Table 4.—Range of sizes of reinforced aerated concrete members manufactured at present in the United Kingdom

Type of Member	Range of Dimension		
	Length	Width	Thickness
Roof and floor units	Up to 20 ft. 5 in.	18 in. to 24 in.	3 in. to 12 in.
Wall units (loadbearing)	7 ft. 6 in. to 20 ft. 5 in.	18 in. to 24 in.	3 in. to 12 in.
Partitions (non-loadbearing)	7 ft. 6 in. to 10 ft.	18 in. to 24 in.	3 in. to 4 in.

Fig. 17.—Small dwelling house using aerated concrete wall and roof slabs.

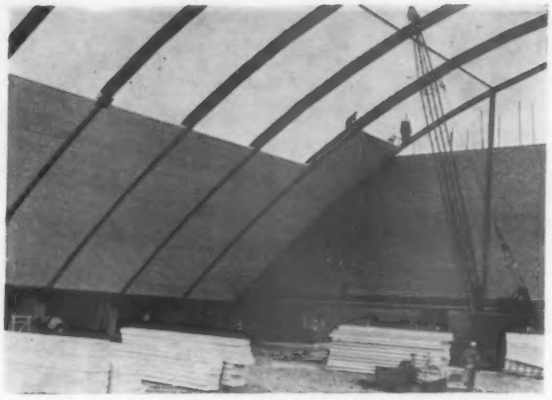

Fig. 19.—Aerated concrete roof slabs used for the construction of a hangar at Bromma Airport near Stockholm.

Conditions of use however tend to vary considerably, not only because of different emphases on purpose, design, finishes, detailing, workmanship, building traditions and durability, but also because of variations in climatic conditions, often accentuated by man-made industrial environments. These are factors the effect of which often cannot be accurately assessed.

Small Dwelling Houses

For single and two storey dwelling houses, reinforced aerated concrete units have been widely used in Scandinavia and elsewhere as loadbearing walls, floor and roof slabs (Fig. 17). Spans are relatively small. Except for early examples containing unprotected reinforcement, rendered units in houses have shown no signs of deterioration.

Industrial Buildings

Aerated concrete flexural members found an early application in industrial roofing (Figs. 18 and 19). In Sweden some 70 per cent of all industrial roofing is made with such elements, not only because of their functional suitability but also because of the saving in funding costs resulting from rapid erection.

Surveys of existing aerated concrete industrial roofs have shown them to be generally satisfactory. It was found also that where conditions were particularly unfavourable, deterioration could be prevented by suitable surface treatment and by proper ventilation of the roof space.

Wall units may be placed horizontally one on top

Fig. 18.—Aerated concrete roof slabs in a textile factory in Denmark.

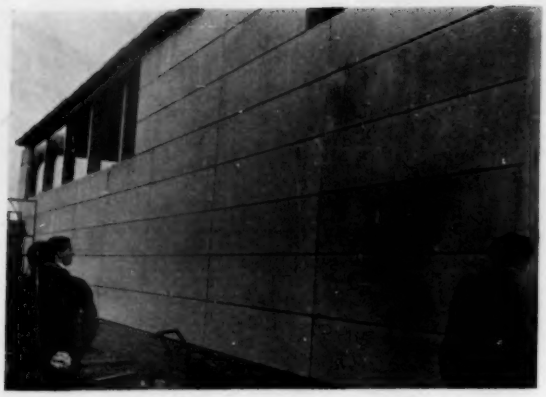

Fig. 20.—Wall built with horizontal wall-units.

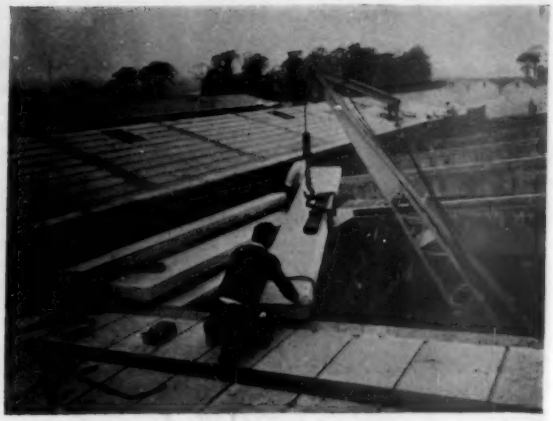

Fig. 21.—Aerated concrete roof slab.

of the other (Fig. 20), following the traditional Swedish timber construction. Alternatively storey-high vertical wall panels are joined along their vertical edges.

Multi-Storey Buildings

For multi-storey buildings aerated concrete slabs are used as roof slabs (Fig. 21), covered by asphalt or roofing felt and also frequently as floor slabs. The use of such light units for party floors seems somewhat difficult to justify on functional grounds alone. Thermal insulation inside an occupied building is not of great importance and aerated concrete is not a good sound insulator. There may be some economic advantage in using the same material and type of construction throughout a large building contract however, particularly in view of the saving in weight, and speed of erection will also count. The floor slabs are usually covered with a cement mortar screed.

Reinforced aerated concrete wall units of storey height, placed vertically, sometimes in the form of sandwich slabs with an insulating layer of foamed polystyrene resin, have found application for the outer, usually non-loadbearing walls and also as partitions, with cast-in-situ reinforced concrete roof slabs and floors. In some tall point blocks near Stockholm and elsewhere in Sweden, the construction is based on a reinforced concrete tower which is first erected on the site with the aid of sliding shuttering.

Fig. 22.—Multi-storey blocks near Stockholm, using aerated concrete wall-slabs with a central reinforced concrete tower.

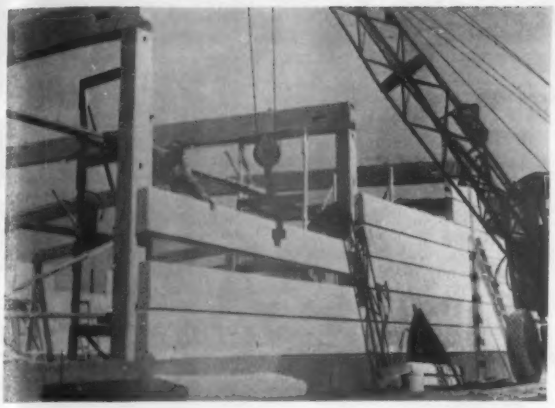

Fig. 23.—Erection of horizontal wall units.

The tower serves as a support for a tower crane, and also as a service duct during construction and after completion (Fig. 22). Structurally, it carries a large part of the design load. The remaining loadbearing structure consists of a reinforced concrete frame or a cast-in-situ panel construction clad with aerated concrete wall panels. Since neither the latter nor the partitions are load bearing, this construction allows a wide variety of floor plans.

Methods of Erection

Many ingenious devices have been developed by manufacturers to increase further the speed of erection with reinforced aerated concrete members and to facilitate construction. Special mobile lifting and levering mechanisms and trolleys have been evolved for roof and floor slabs, to save manpower (Figs. 21, 23 and 24). Devices to speed up accurate aligning have been developed together with improved methods for excluding the weather. Drilling and cutting tools, surface treatments and rendering methods are being tried out and improved.

Fig. 24.—Trolley for placing roof and floor units.

Both as reinforced aerated concrete units and as unreinforced masonry elements, aerated concrete has introduced a standard of accuracy hitherto unknown on building sites. Wall staves now being mass-produced are claimed to have a dimensional tolerance of not more than ·005 in. Due to the grinding process used, opposing surfaces are perfectly parallel allowing the use of glued or even dry methods of erection for walls. This represents a revolutionary change in building methods, and in the attitude of mind required from designers and site workers alike.

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Appendix

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Book Reviews

Rohbaufertiger Stahlskelettbau (*Prefabrication in Multi-Storey Steel Buildings*) by Dr.-Ing. W. Bongard. (Cologne: Deutscher Stahlbau Verband, 1959). 11¼ in. × 8 in., 61 pp.

For years, the German Constructional Steelwork Association has propagated the use of prefabricated elements in the construction of multi-storey steel buildings. This extremely well-illustrated book stems from studies made jointly in 1958 by member firms of the Association and of the Association of Metal Window Manufacturers, under the direction of Dr. Bongard, the object of the investigation being to devise and describe methods of construction which were not only economical but also pleasing from the architectural point-of-view.

In a concise introduction the author contrasts the approach normally made in Europe, where the cost of materials is high, with that in the U.S.A., where labour is expensive. He then describes how curtain walling and light-weight floors and fire-proofing can speed the construction of multi-storey buildings.

The remainder of the book is devoted to ten short chapters, each illustrating a particular form of construction, as follows :—

A. Buildings with visible external stanchions.
B. Buildings with curtain walls.
C. Curtain walls stiffened with light columns.
D. Completely glazed buildings.
E. Buildings with light external columns.
F. Buildings with exposed stanchions and beams.
G. Prefabricated cellular steel floors.
H. Flat construction. Visible skeleton.
J. Cold rolled sections for external sheeting and stanchions.
K. Steel skeletons sheathed with cold rolled sections.

In each chapter there are photographs and excellent details illustrating the particular mode of construction being applied. In consequence, this book should be readily understood by anyone with no knowledge of the German language. G.B.G.

Curve Surveying, by R. B. M. Jenkins. (London : Cleaver-Hume, 1960). 8½ in. × 6¾ in., 184 pp., 35s.

This book is presumably intended mainly for railway engineers, since but little space is devoted to road curves, in so far as they differ from railway curves. It is devoted mainly to the geometry of curves and their setting out, which are well and clearly explained. The theory of transitions is briefly considered, but is based on Shortt, and no mention is made of the work of Leeming and Black in disproof of Shortt's theory, so the value of this section is somewhat reduced. All types of transition curve are described, and tables are given for the clothoid and lemniscate. It is, though, a pity that these tables are included in the text, instead of being grouped at the end for greater ease of reference.

A final chapter explains the Hallade method of realignment of railway curves.

The book is extremely well got up, the printing is excellent, and the diagrams and tables exceptionally clear. J.J.L.

The Design and Construction of Extension to British European Airways Engineering Base at London Airport *

by P. B. **Edwards**, A.M.I.Struct.E., and R. B. Rigg

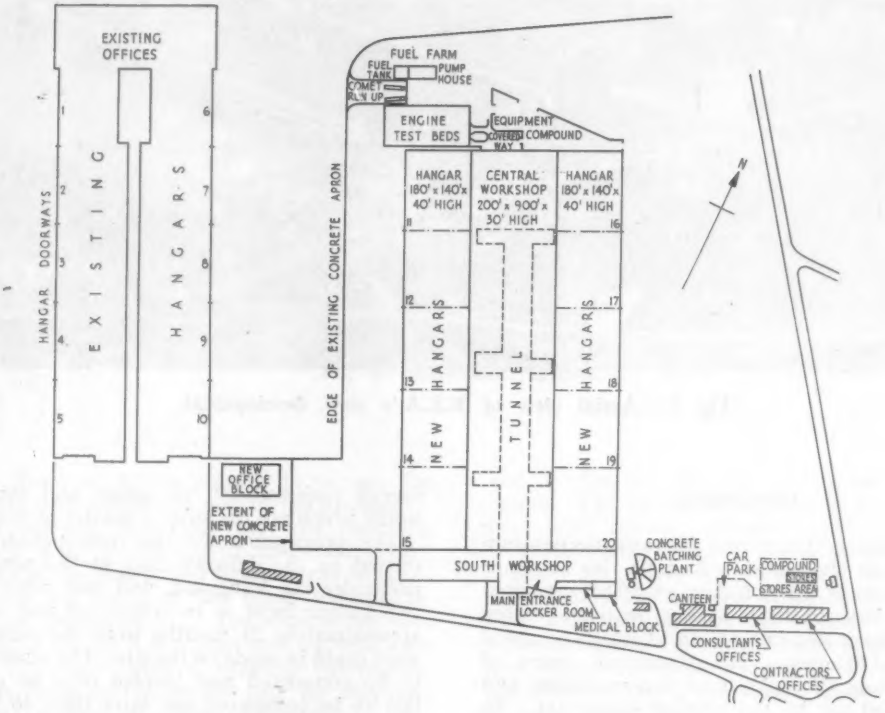

Fig. 1.—Site layout

Synopsis

The subject of this paper is the design and construction of hangars and workshops together with maintenance aprons which form part of the extension to British European Airways Corporation Engineering Base at London Airport.

The design and construction of the works was based on the maximum use of precast concrete construction and the development of the project on these lines was carried out by close co-operation between the Consulting Engineers and the Contractor.

The scope of the project is described in general detail, particular attention being drawn to unusual construction techniques in the field of precast concrete construction. Results of tests carried out throughout the contract are given and compared with design figures.

The paper is sub-divided as follows :

1. Introduction
2. Site Preparation and Sheet Piling
3. Basement
4. Aprons
5. Hangar Construction
6. Central Workshops
7. South Workshops
8. Services
9. Concrete Work and Site Control
10. Programme and Progress
11. Site Organisation
12. Conclusion and Acknowledgements

* *Paper to be read before the Institution of Structural Engineers, at* 11, *Upper Belgrave Street, London, S.W.*1, *on Thursday, 26th January*, 1961, *at* 6 *p.m.*

Fig. 2.—Aerial view of B.E.A.'s new development.

1. Introduction

In 1953, British European Airways Corporation established their permanent Aircraft Maintenance Engineering Base at London Airport. This consisted of ten hangar bays in series of five, with two-storey offices, main stores and aprons, all of which occupied approximately 10 acres. The structural design of the buildings was the subject of a competition and the work carried out by the winning contractor. To meet the maintenance requirements of their increasing air fleet and in particular the Vanguard and Comet IV, B.E.A., decided in November, 1956, that it would be necessary to develop further their maintenance base to the extent of adding a new block of ten hangars and workshops and other ancillary buildings approximately similar in arrangement to their present base but larger in size and with workshops which would afford the maximum flexibility in planning, and give direct access to all hangar bays. The layout shown in Fig. 1, together with the external aprons on either side was considered to be the best solution. Fig. 2 shows an aerial view of the new development.

In order to meet the need for large unobstructed workshops it was decided to provide a basement under the central workshops to house all the ancillary services such as sub-stations, plant rooms, locker rooms and toilets, and furthermore to give access to various parts of the workshop floor, both for plant and personnel.

The Corporation appointed as Consultants the Engineers and associated Consultant team who had acted for them in the construction of their existing maintenance base and in this project they were given overall responsibility to design and superintend the works which are the subject matter of this paper.

The programme for the construction was closely related to the delivery date of the new aircraft, in particular the Vanguard, and this called for the first two hangar bays to be completed and ready for use approximately 21 months from the date at which a start could be made on the site. The remaining hangars to be completed and handed over at intervals, the last to be completed not later than 40 months from the start of the contract. The basement and workshops were required to be completed in sections, the final section handed over at the same time as the last hangar.

The aim of this paper is to show how the project developed and new construction techniques were evolved by the joint efforts of the Consultants and Contractor.

It should be stated that the Contractor viewed the project from the beginning as one in which precast concrete construction should be used extensively. The basic reasons for this were (a) to reduce, on this very large contract, the effects of inclement weather (b) to facilitate a more reliable flow of site work with improved continuity for the various trades (c) to overcome the limited availability of skilled and unskilled labour, largely imported into the area and by utilising the excellent local resources for factory made precast concrete.

2. Site Preparation and Sheet Piling

At the commencement of the Contract, the whole of the site was not available for development, the east portion being still occupied by a fuel storage

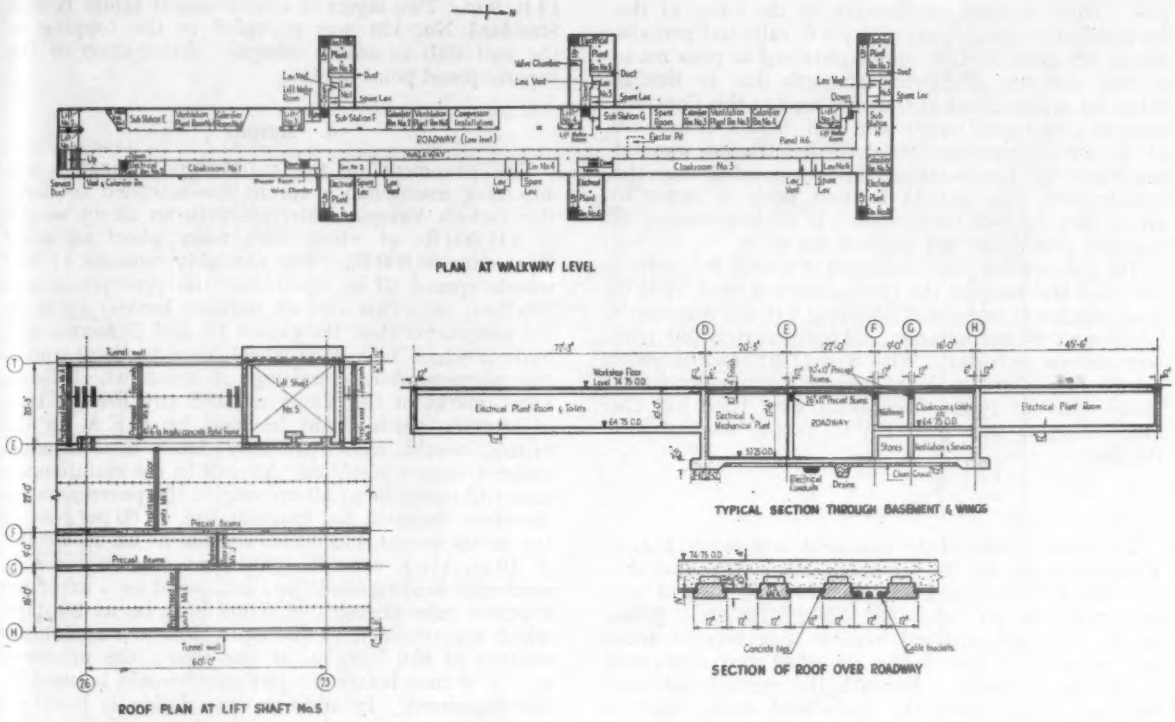

Fig. 3.—Basement layout.

compound. However, it was possible to prepare the site completely west of the east wall of the basement.

The main oversite excavation involved removing approximately 4 ft. of brickearth and sandy clay before the top of the first gravel stratum was reached. The water table was found to vary between 8 to 12 ft. below existing ground level. The excavation was carried out by 2 No. 33 R.B's., one working with face shovel and the other with a dragline equipment, the materials being carted off the site for disposal. Altogether 75,000 yards cube were excavated.

The whole of the excavated site was made up to level with hoggin fill placed in continuous layers which were compacted to 9 in. finished thickness. The material was required before compaction to be brought to a uniform moisture content of 7 per cent differing by not more than 2 per cent (as a percentage of dry weight). This value was varied for different gradings of fill material. Laboratory compaction tests were carried out from time to time on the various gradings of fill to determine the Proctor maximum dry density and corresponding optimum moisture content, whilst site tests to determine the actual moisture content and dry density of the compacted fill were made to ensure compliance with the specification. In addition, plate bearing tests were carried out. An analysis of the results obtained is shown in Appendix I.

It was thought that the excavated gravel from the basement excavation would be suitable for fill, but when examined it showed a lack of cohesion and was therefore only used to a limited extent in the lower layers of the fill.

Trials of various types of compacting machines were carried out, it was finally decided that a 30 cwt. Vibrosol Vibrating plate compactor with one pass, and a vibro-tamper with two passes gave the best results. The fill was brought up to within 6 in. of the final level initially, all subsequent exavation below this level being made into the fill as necessary.

Sheet piling for the basement was commenced immediately the oversite strip had been completed. The piling was carried out to form three separate cofferdams, two sheet piled division walls, each 70 ft. long, being driven as temporary cut-offs in order to commence basement excavation before final completion of the basement.

The sheet piling used along the walls consisted of No. 2 Larssen, 28 ft. in length and driven 10 ft. into the blue clay which underlies the gravel approximately 18 ft. below existing ground level. This piling was tied back to anchor piles, this is illustrated and measurement of anchor loads given in Appendix II. Due to the existence of the fuel compound immediately adjacent to the east wall, tying back of the piling was not possible over a short length of the basement, furthermore any internal strutting within the basement area would have seriously impeded the construction, it was decided that over this length of wall, No. 3 Larssen piles should be driven 27 ft. into the blue clay and that they should act as a free cantilever.

During the course of excavation within the cofferdams, frequent measurements of the lateral movement of the sheet piling were made ; the maximum horizontal movement however of the tied piles never exceeded $\frac{1}{2}$ an inch. This was not the case in the cantilever piles, for at one stage during operations a horizontal movement of $2\frac{1}{4}$ in. was recorded along a section of the wall. Various tests were carried out to ascertain the reason for this, but it was finally thought to be attributable to lack of friction at the joints of each

pile. There is some controversy on the value of the friction in the clutches of sheet piled walls and perhaps due to the good friction that is obtained in piles made in this country, additional strength due to this is taken for granted, but as the piles used in this Contract were of continental origin and had greater clearances at the joint it was considered the full friction was not developed in this instance. It was decided that the cantilevered piles should be tied back in order to arrest any further movement. It so happened that this was possible at this stage of the work.

The pile driving plant consisted of a 43 R.B. pitching the piles and moving the gates, followed by a 22 R.B. from which was suspended a Mencke S.B. 180 Automatic Compound steam hammer. Approximately 30 piles were driven each day. With 8 to 12 ft. head of water behind the cofferdam, leaks became a serious problem. Many different types of caulking were tried but the most effective was ¼ in. P.V.C. tubing worked into the joints.

3. Basement

The arrangement of the basement is shown in Fig. 3. The excavation for the basement was taken generally into the blue clay and in order to take care of any leakage of the ground water through the sheet piling to the underside of the basement a perforated drain laid in selected gravel was provided and connected to the main ejector. Beneath the ground slab and running parallel with the perforated drain there is the main trunk main for the foul drainage which finally delivers into the ejector pit. All the central workshop and toilet drains deliver into this main.

The floors and walls were constructed of in-situ concrete, the latter being waterproofed by three coats of asphalte tanking. As previously mentioned, a fuel compound was sited immediately adjacent to the basement and it was found that the ground water was contaminated with fuel oils. This presented a serious threat to the asphalte tanking and many enquiries were carried out to find a suitable protection to the asphalte. Three coats of P.V.C. paint in the end proved to be the most effective. Each coat was pigmented a different colour to ensure proper coverage. Some difficulty was experienced when the paint was applied to the horizontal surface under the access wings and the lift pits, due to bubbling of the membrane and slowness of drying caused by the moisture seeping through the concrete blinding. This was overcome by applying the P.V.C. paint to a layer of 'Fibroin' felt.

Recognising that the in-situ work of the basement floor and walls would slow down progress, the internal structure and roof were designed to achieve rapid construction. The columns were standardised and cast in-situ. All beams were precast to the underside of the slabs and the slabs consisted of precast prestressed units with infilling concrete tiles and in-situ topping. The mezzanine floor was designed for a superimposed load of 60 lb/sq. ft. and the roof of the basement for superimposed load of 2½ cwts. per square foot. The precast main beams were designed to carry the units and wet concrete initially, and the superimposed load after hardening of the in-situ topping.

The centre lines of columns were taken through to the workshops at 60 ft. centres for the support of the workshop roof. Access to the lift frontages was made possible by substituting portal frames of up to 45 ft. span in place of the usual standard 15 ft. column spacing. The precast prestressed units required propping at one third point where the span exceeded 14 ft. 9 in. Two layers of reinforcement fabric British Standard No. 124 was provided in the topping of the roof slab to ensure adequate distribution of the superimposed point loading.

4. Aprons

The pavement of the maintenance hangars and adjoining maintenance aprons are intended to carry the Vickers Vanguard aircraft with an all-up weight of 141,000 lb. of which each main wheel assembly represents 66,000 lb. The assembly consists of two wheels spaced 27 in. apart and the tyre pressure is 100 lb/sq. in. This aircraft imposes heavier loads on the pavements than the Comet IV and Britannia 312, both of which have four wheel (dual tandem) undercarriage assemblies resulting in lower wheel loads. Later marks of the Vanguard and any heavier type of aircraft which might be used by B.E.A. in the future, would most probably have dual tandem undercarriage assemblies. Aircraft in the maintenance area will rarely be at all-up weight, the pavements are therefore designed for frequent use at 80 per cent of the all-up weight and occasional use at all-up weight. A 10 in. thick concrete pavement was required to meet edge loading conditions and called for a minimum concrete cube strength of 5,000 lb/sq. in. at 28 days, which was considered to be equivalent to a modulus of rupture of 610 lb/sq. in. at one year; the minimum length of time before the pavement would be used by the Vanguard. In addition, the design is based on a modulus of subgrade reaction (k-value) of not less than 800 lb/sq. in. This design gave a factor of safety of 1.7. Consideration was given to dowelled joints with a corresponding reduction in thickness of the slab, but it was felt that the construction of a slab the same thickness throughout was more economical and quicker to construct.

The hangar floors are constructed as compound slabs in order to make provision for underfloor heating. Originally the make up of the slab was 3 in. blinding to support the heating coils followed by 2½ in. screed to cover and protect them and finally 8 in. thick quality concrete slab with integral granolithic finish. This construction proved a tedious operation and it was decided to combine the 2½ in. screed and 8 in. slab into one 10½ in. slab using the external pavement grade of concrete. The bay sizes are generally 12 ft. by 12 ft. with construction joints running in the length of the hangars and dummy joints across the width at 12 ft. centres, the latter being saw cut. Expansion joints were provided between each hangar bay and also around the heating ducts and adjacent to enclosing walls. Special care was taken to see that the joints in each component slab were identically placed to ensure that the slab would crack at the joints rather than within the slab. For details of joints see Fig. 4.

The hangar floors are split up by heating ducts and service pits (see typical arrangement Fig. 4). The ducts between service pits were precast in units 8 ft. 9½ in. in length. The construction procedure was as follows:

Two inches of concrete blinding was laid to the correct level minus ½ in. approximately; prior to positioning of the units a wet mortar bed was laid over the blinding, approximately 1 in. thick; finally, units were laid on to the mortar bed. The ends of each bottom slab of the units were jointed with mortar and the walls concreted together. This method of construction proved extremely rapid and saved a lot of formwork and extensive site concreting. The construction around the service and drawpits was carried out in-situ.

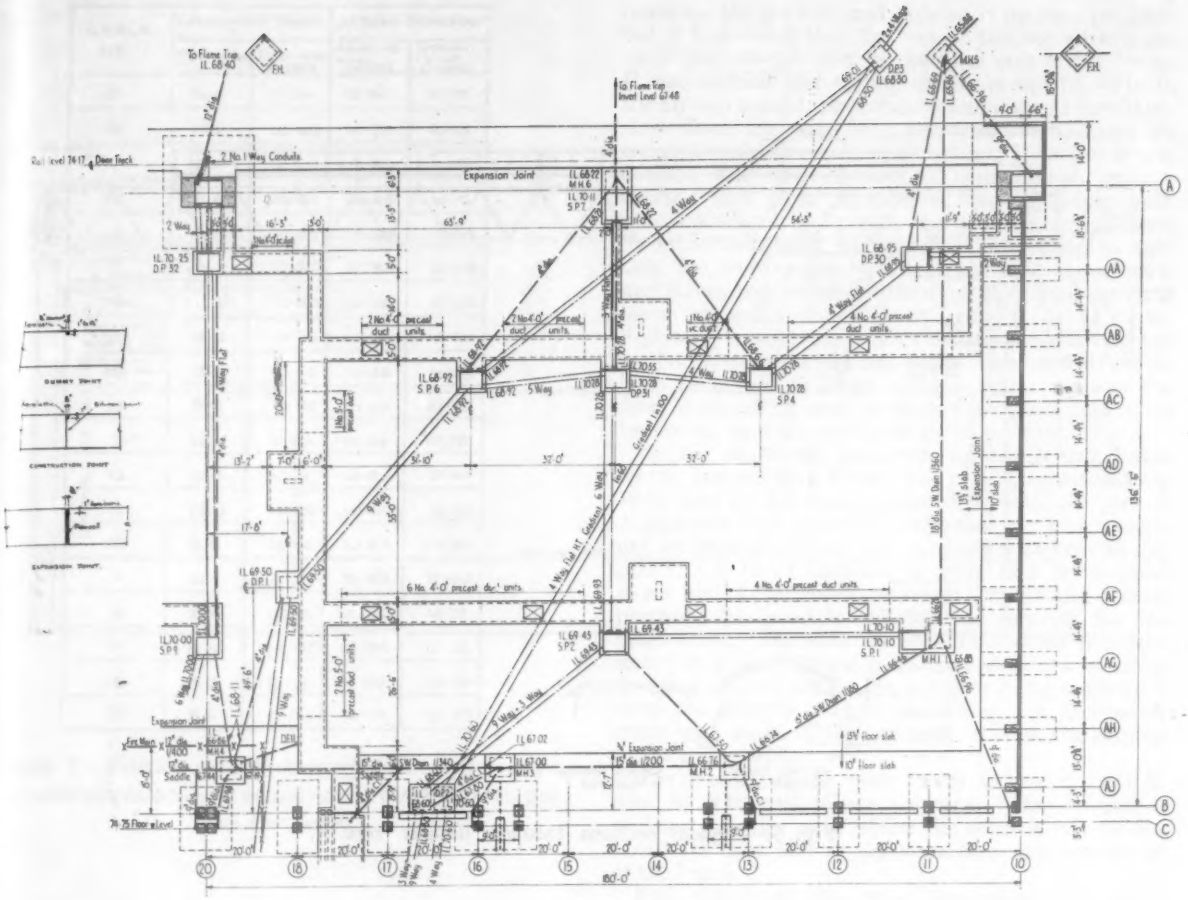

Fig. 4.—Layout of hangar floor.

5. Hangar Construction

The basic requirements for the hangars were that each bay should be 180 ft. long and 141 ft. 3 in. in width, measured between inside face of doors and rear wall with a main door opening 168 ft. wide by 40 ft. high in the clear, and a 3-ton overhead travelling crane spanning across the width of the hangars. Furthermore as much natural light as could be obtained was to be provided (see Fig. 5).

The choice of a hollow box member in prestressed concrete for the main hangar door beam was made for the following reasons :

(a) It provides cladding as well as structure with minimum maintenance.
(b) The 3-ton crane structural depth and door canopy height together demand a member at least 13 ft. 6 in. deep.
(c) The box section is the best structural member to carry the torsion from crane and door canopy.
(d) Considering only the cost of the main beam itself the concrete box is considered to be no more costly than any other type of member that would suit the planning.
(e) It was required to maintain similar architectural elevation and to transfer the lettering from the east face of the existing base to the new east elevation so that a fascia of approximately 20 ft. in depth was required.

The choice of a tubular space frame for the secondary beams in preference to prestressed concrete beam or any other form of beam was made on the grounds that (a) aesthetically it met the overall requirements for lightness in appearance ; (b) its own weight would only be approximately 6 tons as compared to 40 tons in the case of a solid concrete member or 20 tons for a latticed concrete member ; (c) by reason of its lightness the erection problem would be easier.

The remaining parts of the flat roof and gutters are covered with pressed aluminium roof decking, fire retarding insulation board and three layers of bituminous roof felting. The hollow box main beam is 14 ft. deep by 6 ft. 6 in. wide with a 4-in. preformed camber, and consists of a series of precast diaphragms at 10 ft. centres, every alternate one being extended to support the canopy and head track for the doors and located to take the direct load from the secondary roof beams. The bottom slab of the box section is 5½ in. thick, top slab 8½ in. thick and walls 5 in. thick and were cast in situ between diaphragms and precast end anchorages. On completion the beam was post-tensioned with 38—12/0·276 Freyssinet cables which were threaded before the concreting operation through unitube ducting placed in walls and soffit, see Fig. 6. Stressing of the beam was carried out in two stages, cables 1 to 15 inclusive were stressed after the main beam had been cast, when the concrete had reached

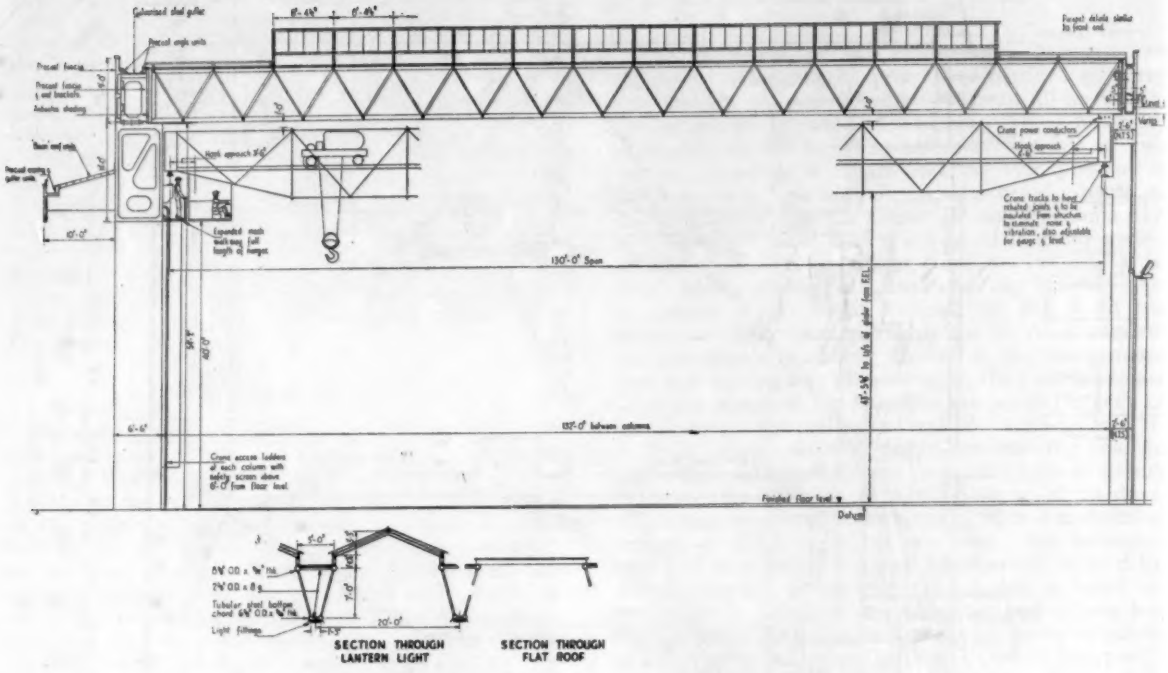
Galvanized steel gutter
Precast angle units
Precast fascia
Precast fascia c and brackets
Asbestos sheeting
Hook approach 3'-6"
Expanded mesh walkway full length of hangar
Crane power conductors
Hook approach 2'-0"
Crane tracks to have rebated joints c to be insulated from structure to eliminate noise & vibration, also adjustable for gauge & level.
130'-0" Span
137'-0" between columns
Crane access ladders at each column with safety screen above 6'-0" from floor level.
Finished floor level
Datum
SECTION THROUGH LANTERN LIGHT
SECTION THROUGH FLAT ROOF

Fig. 5.—Typical sections through hangar roof.

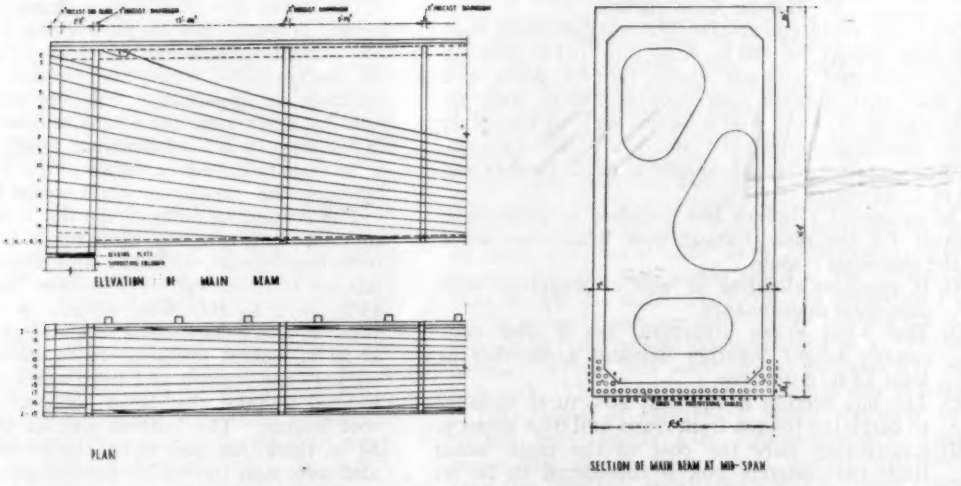

Fig. 6.—Main beam. Cable layout.

CABLE No	CALCULATED VALUES		ACTUAL EXTENSIONS	
	% FRICTION	EXTENSION INCHES	HANGAR SIDE INCHES	APRON SIDE INCHES
8	14·0	10·44	10·61	10·79
9	13·5	10·46	10·89	10·76
7	14·0	10·44	10·59	10·84
10	11·6	10·50	10·68	10·72
6	14·7	10·48	10·89	10·64
11	10·7	10·54	10·88	10·68
5	14·8	10·50	10·74	10·62
12	10·5	10·56	10·74	10·60
4	15·0	10·54	10·46	10·58
13	9·8	10·60	10·67	10·79
3	14·9	10·54	10·74	10·73
14	8·5	10·74	10·86	10·60
2	15·0	10·54	10·40	10·50
15	8·5	10·74	10·83	10·94
1	22·0	9·0	9·59	9·60
16	8·5	10·74	10·86	10·77
17	8·5	10·74	10·99	10·66
18	8·6	10·74	10·92	10·73
19	8·5	10·74	10·73	10·89

Table 1.—Friction : Table comparing calculated values of cable extensions with actual values obtained averaged over the beams.

the required transfer stress, and the tubular secondary beams had been erected. Cables 16 to 19 were stressed after the military trestling supporting the formwork had been removed and 'Bison' precast roof units and precast fascias of the canopies were fixed in position. The cables were stressed to 73 tons/sq. in. by jacking from both ends and later grouted with neat cement and sand with an 0·45 water-cement ratio using an expanding additive.

The procedure for stressing the main beam was as follows :

(1) The Freyssinet jacks were calibrated on a test rig.
(2) Stressing commenced at both ends of the beam and the gauge reading brought up to 2,000 lb/sq. in. on both jacks A and B.
(3) The pressure on jack B only was increased until there was a change in the reading on the pressure gauge of jack A.
(4) By taking these two readings and applying them to the following formula

$$F = (1 - \sqrt{A/B})\,100$$

an assessment of the total friction to be encountered on the cable was obtained.
(5) Jack A pressure increased to equal that of jack B.
(6) Both jacks increased to the calculated gauge pressure for a force of 73 tons/sq. in. in the cable.
(7) Cables anchored.

The end reactions of the beam are transmitted through bearing plate assemblies to the supporting columns. The beam is stressed vertically down through the bearing plates with two 12/—0·276 Freyssinet cables anchored in the lower part of the column. Consideration was given to precasting the main beam on the ground, and jacking it up into position, but it was found that the cost of jacking worked out more than 75 per cent of the total cost of the beam. It was realised that casting the beam in-situ, 40 ft. in the air was a major undertaking and it could inevitably slow down the rate of construction, obtainable by extensive use of precast construction. However, it was decided that this method of construction, once a routine had been established, would prove both economical and fit in with the programme governed by other associated works. It is interesting to note that the construction time for the first main beam was eleven weeks and by the time the fifth beam had been reached this had been reduced to 5½ weeks. Following a work study on the military trestling the method of construction was slightly modified to enable it to be moved in larger sections which reduced the total dismantling and erection time from four days to one day and the total cost by 74 per cent.

Special shuttering was made for the beams, based on the Symons Wall Form. The panels, approximately 10 ft. long by 2 ft. wide, spanned between the precast diaphragms and provided a two-foot lift which could not be exceeded due to the congestion of prestressing cables. A 14/10 Stothard & Pritt high level diesel mixer with a road machine weight batcher and cement portasilo was used for mixing the concrete for the main beam. The concrete was transported from the mixer in a dumper which discharged into a specially designed skip to enable the concrete to be discharged from the dumper without shovelling. A Paymaster crane with an 80 ft. jib was used to hoist the skip up to the main beam.

The load from the main beam is carried on 6 ft. 6 in. by 3 ft. cast in-situ columns. One column is stiffened by 9 in. walls whilst the other is free to flex about its base and so accommodate any movement of the beam.

Full details of the actual extensions and forces applied to the cables were kept for each beam. An analysis of these details shows that the average extension for each particular cable over the range of 10 beams comes very close to the calculated extension based on Cooley's Method (See Table 1). Also given in the Table is the percentage friction corresponding to the extension.

The fascias to the canopy and above the main beam were precast in 20 ft. lengths and bolted to the main beam assembly. The fascia above the main beam was the final component to be erected and followed the completion of the roof. This enabled a cast in-situ seating for it to be formed, the top of which was made truly horizontal thus masking the residual camber in the main beam.

The rear wall of the hangar is split up into bays by columns at 20 ft. centres ; the infilling being 9 in. brickwork up to 33·00 level above hangar floor and patent glazing up to the underside of the capping beam at 50·00 level approximately. In order that future aircraft can be accommodated in the hangars a 40 ft. wide by 20 ft. high opening was required in the centre of the rear wall of each hangar to allow the nose of aircraft to protrude through into the central workshops.

This opening was achieved by carrying two columns of the 20 ft. module on a beam 26 ft. above floor level spanning 42 ft. to two extra columns of the 20 ft. module. The columns generally were braced by a precast beam at the bottom of the glazing and again by a hollow capping beam at the top of the glazing. For general arrangement of this construction and details see Fig. 7.

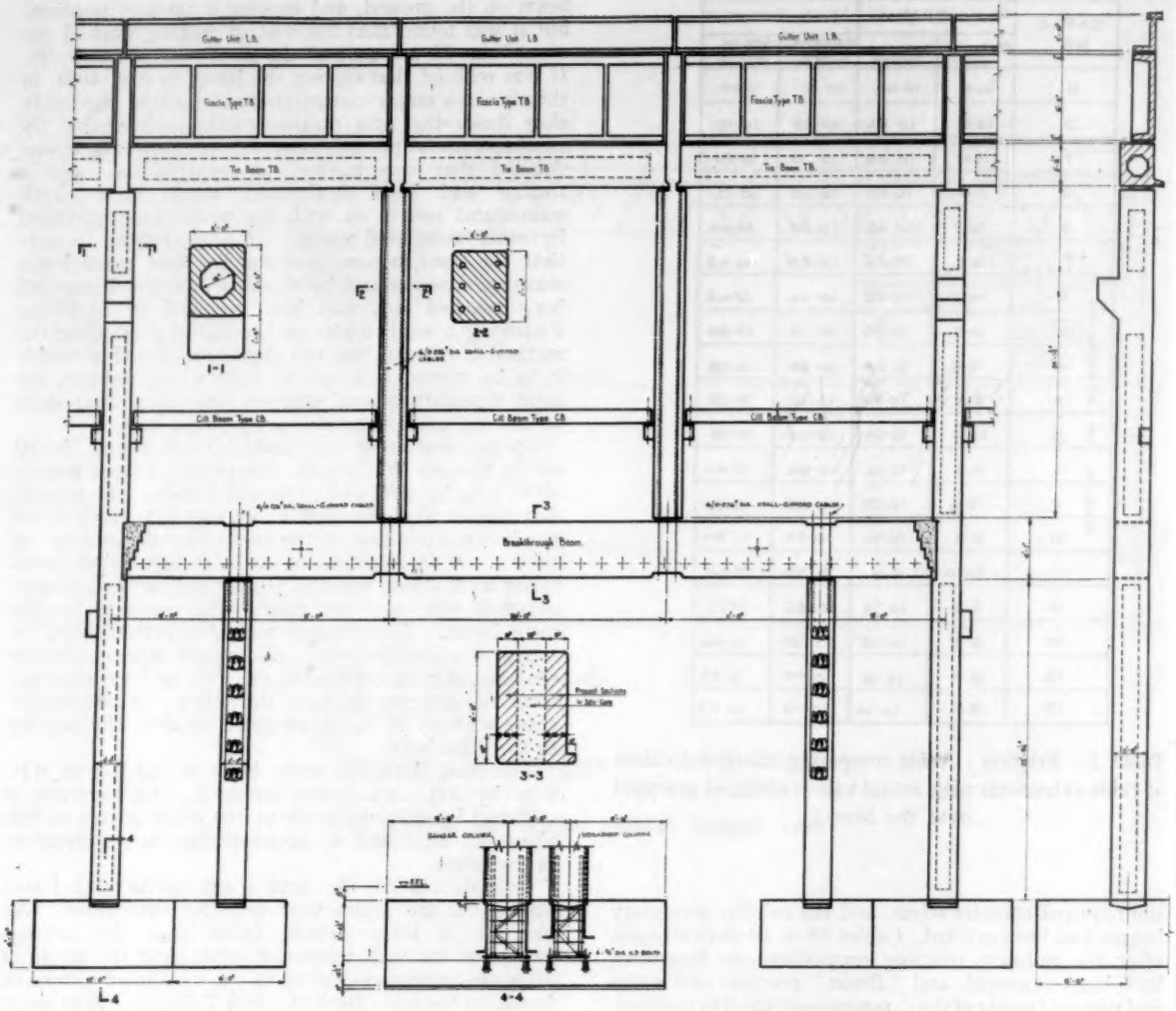

Fig. 7.—Rear hangar wall—precast construction.

The rear hangar wall columns 60 ft. 9½ in. in height were cast in one length and provided with hollow cores to reduce their weight. They are 30 in. by 24 in. up to 51 ft. above base reducing to 11 in. by 15 in. for the remaining height. This reduction was made to accommodate the capping beam and rear wall precast fascia panels. The columns are provided with corbels to support the crane gantry beams.

The foundation bases for these columns also accommodate the 30 in. by 30 in. columns of the central workshop, there being a 12 in. gap between faces of both columns. A design for these bases was evolved (see Fig. 7) in which the columns were initially erected in a similar manner as a steel stanchion; the base being finally cast after the column has been positioned and plumbed.

The method of erection of these columns may be briefly described as follows :

Prior to erection the lower slab of the foundation base 1 ft. 3½ in. thick was cast with bottom and top reinforcement into which were cast four ¾ in. dia. H.D. bolt assemblies. The columns arrived on the site with the main reinforcement protruding from the base of the columns; into the solid core were cast 4—1½ in. dia. bars tied with reinforcement links and latticed with steel angles. The ends of the vertical bars were welded to a M.S. base plate 1 in. thick in which 13/16 of an inch dia. holes had been drilled to receive the H.D. bolts.

Levelling and lining was carried out as follows :—

The distance from the crane corbel to the column base plate was measured and a 6 in. by 6 in. steel plate bedded into the lower column base slab at the correct level. Centre marks for plumbing were made on two faces of each column. In addition the centre line of the column would be transferred and marked on the column base plate. Sand and cement spots would be put down in the in-situ column base in a central position on each face of the column. These would be marked with the centre line of the column.

The column would be lifted by an 802 Lima crane and swung into position over the in-situ base. With the lines on the column base plate held in line with the

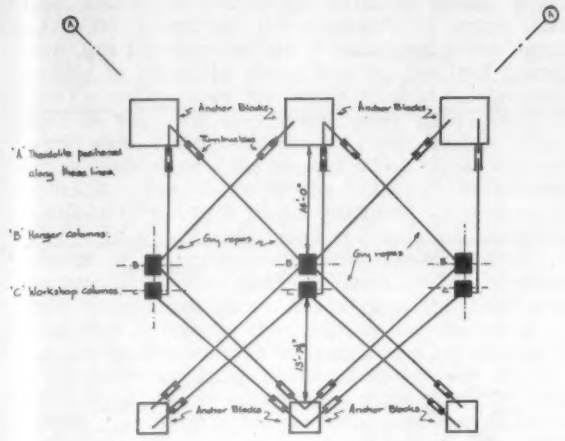

Fig. 8.—Position of guy ropes.

marks on the sand and cement spots the column would be lowered on to the 6 in. by 6 in. steel plate.

After plumbing by spirit level the H.D. bolts would be tightened up and guy ropes attached and taken to the anchor blocks and tightened with turn buckles. The crane would then be released to erect another column. Meanwhile two theodolites set up into line with the guy ropes would be used to plumb the column, necessary adjustments being made by the turn buckles. Finally any packing under the base plate was carried out. The arrangement and details of the base and position of guy ropes are shown on Fig. 8.

In practice it was observed that the smaller of the two columns could be erected and plumbed accurately in 20 minutes.

The 48 in. by 30 in. beam over the 40 ft. wide clear opening in the rear wall was found to be too heavy to precast in one, it was therefore designed to be constructed in three vertical slices, the outer two being precast sections 48 in. by 9 in. and the inner cast in-situ 48 in. by 12 in. to link each slice together. The beam sits on short columns either side of the opening and spans over these to link up with the adjacent main columns on the 20 ft. module. The beam is stressed down onto the short columns by 6—12/0·276 Udall-Gifford cables. Due to the uplift at the end of the beam where they join the main columns the precast sections were toothed in order to transmit through an in-situ joint the shear force to the main columns. Holes were left in the precast sections from which the soffit shutter could be supported. The reinforcement of each slice is linked together as well as the continuity bars from the main columns by ' U ' shape stirrups placed from the top and bottom of the beam and lapped at the sides. This beam supports two columns of the 20 ft. module and was designed to transmit the wind load by torsion as well as the vertical loads to the short columns on either side of the opening. The columns are post tensioned onto the beam by 6 — 12/0·276 dia. Udall-Gifford cables, two of which are anchored in each precast unit and two in the in-situ section.

At the top of the rear wall a hollow capping beam 30 in. by 30 in. is provided below the level of the secondary roof space frames. Precast ribbed channel shaped fascia units are mounted on this beam and a precast combined gutter and coping unit is bolted above it. Precast beams supporting infilling brickwork and glazing are bolted to the main columns at various levels. Throughout the rear wall precast concrete construction a maximum tolerance of $\frac{3}{4}$ of an inch was permitted in any direction and all attachments to the construction including the glazing was designed to accept this tolerance.

The north and south walls of the hangars are largely glazed, the lower 22 ft. and side bays being 6 in. in-situ concrete walls. The walls are stiffened by columns at 14 ft. $4\frac{1}{2}$ in. centres cantilevered from their bases, a hollow capping beam is provided at the top of the wall on which the purlins of the roof construction slide as the wall deflects under wind pressure. The columns which are 60 ft. $2\frac{1}{2}$ in. in height are designed to reduce the horizontal deflection rather than to carry direct load. They are 36 in. by 20 in. at the base reducing by 6 in. and 12 in. at two levels to 24 in. by 20 in. at the capping beam. To reduce the weight to 16 tons for erection purposes the columns are cast with two hollow cores for two-thirds of their height and a single hollow core for the rest of their height. Foundation bases for these columns were provided with tapered sockets into which the tapered end of the columns were inserted and concreted around. It was found when designing these foundations that where large bending moments occur in the column there are limitations on the use of this type of construction, due to large shear forces requiring a high percentage of reinforcement in the tapered end of the column.

The tubular steel secondary roof beams are designed as space frames 7 ft. 6 in. overall in depth, spanning 133 ft. and placed on a module of 20 ft.; they weigh only 6 tons. A preformed camber was built into the frames to give adequate roof drainage falls after all the roof load had been applied. Between the frames span monitors of joist section supporting patent roof glazing. Both these and the space frames are shot blasted and metal sprayed with 0·005 in. of aluminium.

The frames were fabricated in sections in the works and transported to the site following the aluminium treatment. A complete frame comprised 3 units which were welded together on site and the joints metal sprayed.

6. Central Workshops

A variety of structures was considered for the roof of the workshop on the basis of the clients' requirements that (a) the 200 ft. width of the workshops should be unobstructed except for a row of columns down the centre and (b) that monorails would be required to be affixed to the underside of the roof structure.

The final choice of a northlight tubular steel roof frame on a module of 20 ft. (see Fig. 9) was made because it was aesthetically consistent with the hangar construction and in addition represented the best solution from an erection point of view. The choice of a 60 ft. module for the centre row of columns was a compromise between the basement grid at 15 ft. centres, the workshop frames at 20 ft. centres and the desire to give a reasonably open grid for flexibility in the workshops.

Each 20 ft. wide northlight frame consists of a north and south tubular lattice girder with a common top boom at the apex. Three complete frames are joined by the members of the hipped ends of the northlight frames with top booms added outside the cladding providing a sloping lattice spanning 60 ft.

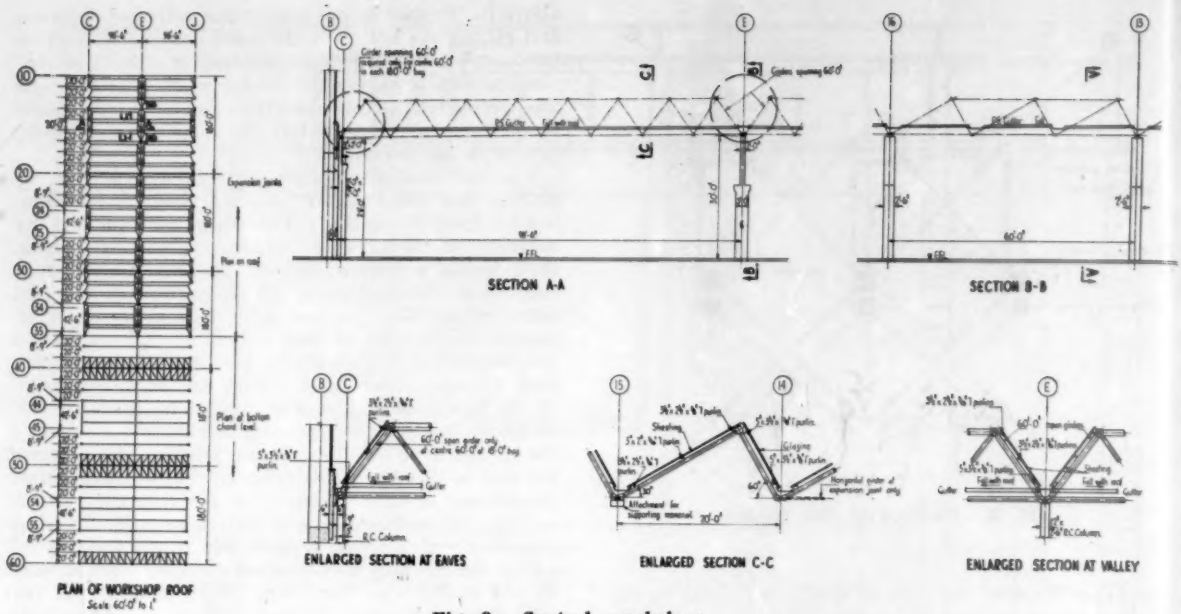

Fig. 9.—Central workshop.

The bottom booms of the north and south girders were joined by a single 1½ in. dia. bolt through gusset plates welded at 90 degrees to the tubular boom. This bolt was provided at the point of intersection of the centre lines of the frames and interleaved between these plates was a third plate which formed the point of suspension from which the monorail system was supported. The construction of the lattice frames was carried out on the site. The erection of the roof frames was carried out in sets of three north-light units and these were welded together in the air. The erection was carried out by two electrically operated derricks on rails. The north lattice girder is clad with patent glazing whilst the south girder is clad with aluminium decking and a 3-layer bituminous felt covering. Horizontal wind bracing was provided in the end frames of each 180 ft. of workshop where an expansion joint to conform to the hangar module was provided. At the time the roof structure was designed an allowance for monorails was made for two three-ton loads on any frame at 20 ft. centres. After the roof frames had been fabricated, the design requirements were changed to two lanes of crane track in every 100 ft. width of workshop on which

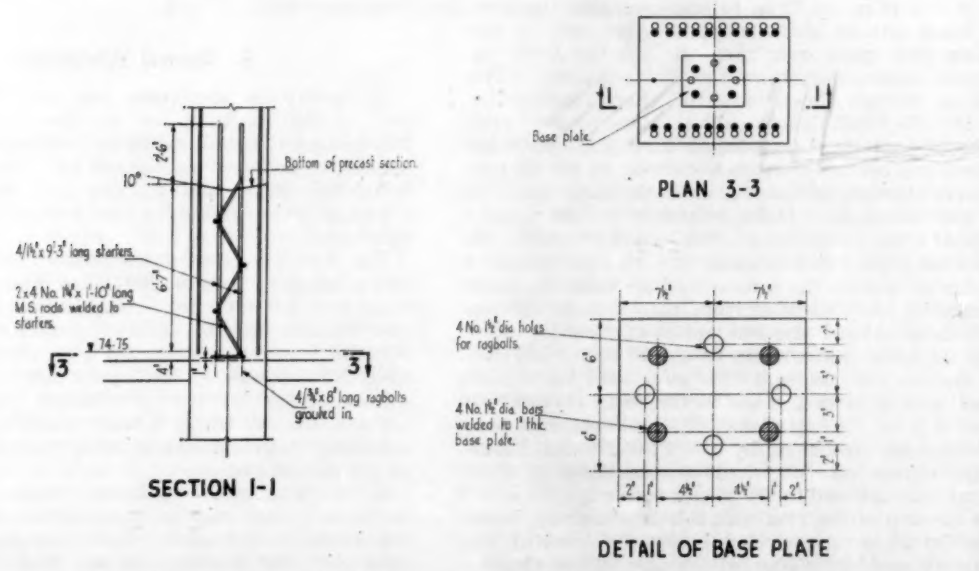
Bottom of precast section
4/1¼" x 9'-3" long starters
2 x 4 No. ¾" x 1'-10" long M.S. rods welded to starters
4/⅞" x 6" long ragbolts grouted in
SECTION 1-1
Base plate
PLAN 3-3
4 No. 1¼" dia. holes for ragbolts
4 No. 1¼" dia. bars welded to 1" thk. base plate
DETAIL OF BASE PLATE

Fig. 10.—Construction of central workshop columns.

3-ton and 1-ton underslung travelling cranes would run. By restricting the approach of cranes and conditions for operating the 3-ton cranes it was found possible to carry the direct load on the roof frames from the crane tracks, but it was felt that the dynamic effect of the long travel and cross travel traction forces were undesirable on a rigid welded frame and in particular on the bracket which would support the track. The choice of the 'Demag' underslung travelling cranes with tracks supported by suspension bolts, with a ball and socket end to allow complete freedom of movement, overcame the difficulties of accommodating the traction forces. This flexible crane system has many advantages, the chief one being that it reduces the tendency for the crane to crab and furthermore the dynamic forces are cushioned by the pendulum action of the tracks (see Fig. 11).

The 24 in. by 24 in. columns supporting the roof adjacent to the rear hangar wall were erected with the hangar columns as previously described. The central row of columns 30 in. by 30 in. were also precast and provided with a lattice foot similar to the hangar columns, starter bars from the basement structure lapping with the reinforcement in these columns. Only the bottom four feet of the column was cast in-situ. The lower part of the precast column was wedge shaped to allow the air bubbles to escape while the in-situ concrete was being vibrated and a box was constructed at two faces of the column to ensure complete compaction. This led to a small corbel being cast and this was trimmed off while the concrete was still green (see Fig. 10).

The north wall of the workshops consisted of precast concrete columns at 16 ft. 8 in. centres with precast concrete purlins 9 in. by 9 in. at 5 ft. 5 in. centres bolted on to the columns. The purlins support aluminium 'Snaprib' sheeting.

The workshop floor consists of a 6 in. thick lower slab with a 4 in. upper slab to contain all service runs, electrical ducting, etc. The upper slab has a monolithic grano finish.

7. South Workshop

This workshop which is 100 ft. wide by 500 ft. in length is again a combination of tubular steel roof framework and precast concrete supporting members. Along the north side adjacent to the hangars and central workshops 24 in. by 24 in. precast concrete columns are sited at 50 ft. centres with 33 in by 24 in. hollow precast capping beams. The tops of these beams are notched out at their ends to enable an in-situ link to be made with the columns. A similar arrangement with 18 in. by 15 in. precast concrete columns at 16 ft. 8 in. centres forms the outer face of the building, the capping beams to these columns being 24 in. by 24 in. solid in section and linked to the columns in a similar manner to those on the north side. Precast concrete purlins 9 in. by 9 in. in section and notched on top to receive the cladding clips are bolted to the columns at 5 ft. 5 in. centres. (See details on Fig. 12). At the east end of the workshops there is a 50 ft. wide by 25 ft. high clear opening which is provided with an electrically operated sliding door. The west end is a temporary end with steel stanchions embedded into concrete foundations; these support steel purlins on which the cladding is fixed. The tubular steel welded roof is a series of monitors on a 16 ft. 8 in. module. The main lattice girders are 9 ft. overall in depth with hipped ends and a preformed camber is built in to give roof falls. Tubular steel purlins support the roof decking and

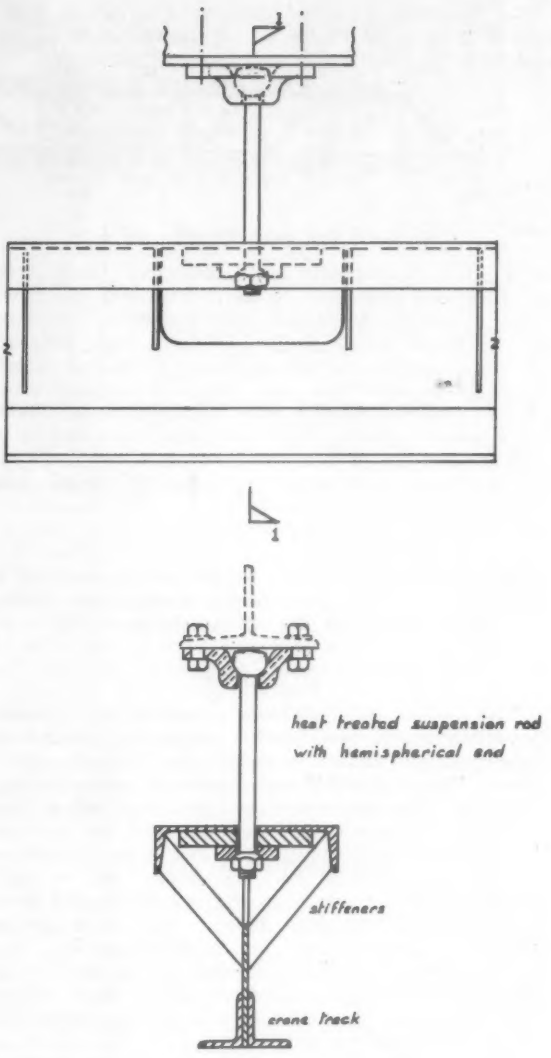

Fig. 11.—Suspended crane track.

the sides of the girders are patent glazed. The girders were fabricated in sections and assembled on site, the purlins and wind bracing being bolted to the girders. At each node point a 5-ton point load was taken into account in the design to allow for underslung cranes.

The external cladding to the workshops consists of 'Snaprib' aluminium sheeting with an asbestos sheet inner lining insulated with glass fibre.

Centrally placed in front of the south elevation is a single storey main entrance hall giving access to the basement. The roof over the hall consists of a main rectangular tube steel lattice girder spanning 62 ft., the top flange supporting on one side, secondary lattice girders at 14 ft. centres and the bottom flange on the other side supporting 6 in. by 4½ in. by 20 lb. R.S.J's. at 9 ft. 3 in. centres, both of which carry the aluminium roof decking and 3-layer bituminous felt covering. The external side of the main lattice

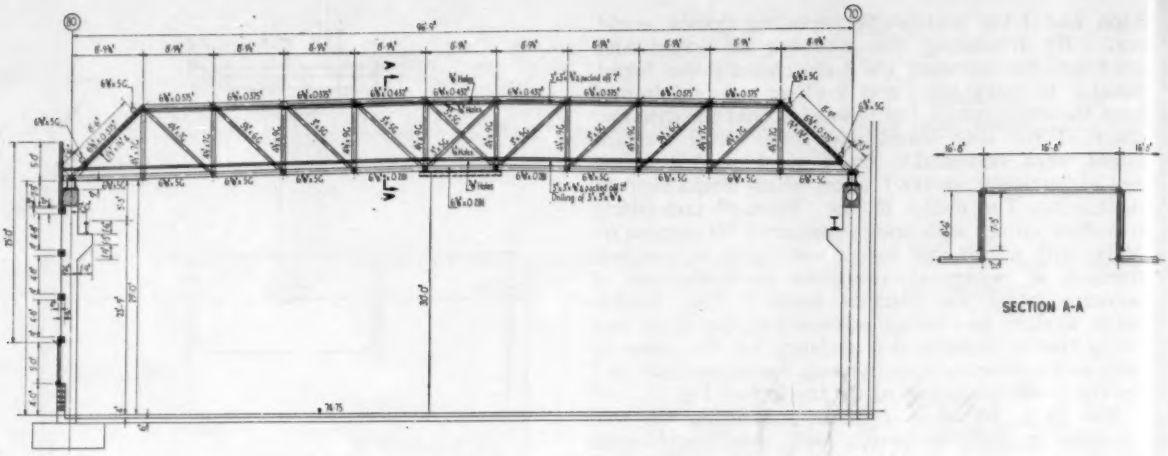

Fig. 12.—Cross section through south workshop.

is patent glazed. To the west of the entrance hall is the main entrance and tubular steel canopy giving external access to the lifts serving the basement.

8. Services

The services, with all their associated plant rooms, formed a major part of the project, and called for careful consideration in conjunction with the structural work. The mechanical services include space heating, hot and cold water services, gas, ventilation, high and low pressure compressed air, and fire services. The electrical services include 11 kv ring main extension to six new sub-stations, the fitting out of eight electrical plant rooms serving the hangars and workshops, and the running of power supplies to service pits in the hangar floor; in the workshops by means of an overhead busbar trunking system and in the floor, through the underfloor metal duct system laid in the form of a grid with outlets at approximately 20 ft. centres in both directions.

The underfloor heating coils in the hangars involved the use of approximately 20 miles of mild steel pipe. The heat source is provided by a central high pressure hot water boiler house which was erected adjacent to the base recently by the Ministry of Civil Aviation. The mains, a total length of approximately 3,000 ft., were laid in precast concrete ducts, 5 ft. 6 in. by 3 ft. internal reducing to 4 ft. by 3 ft. internal over 770 ft. run of duct. The precast construction of this duct was similar to that in the hangars.

In view of the complex nature of the services, all sub-contractor's details of pipe runs and cabling were co-ordinated on to one set of drawings to ensure as far as possible that no one service ran foul with another, and also that combined supports could be provided, rather than each sub-contractor supplying his own. A great deal of time and discussion was involved in this co-ordination work, but it was essential to the orderly and speedy installation of the services.

9. Concrete Work and Site Control

Altogether, approximately 65,000 cu. yds. of in-situ concrete was placed on the site; 45,650 sq. yds. of apron concrete pavements and 50,000 sq. yds. of hangar and workshop pavements. 1,200 tons of steel reinforcement was used in the in-situ concrete work, and 166 miles of 0·276 dia. prestressing wire was used in the prestressed concrete work. The main concrete plant was a Winget 1 cu. yd. batching plant with bulk cement silo and boom scraper.

A site laboratory was set up on the site to enable soils and concrete tests to be carried out. Details of the tests carried out on the concrete work and their frequency are shown in Table 2. Histograms showing

ROUTINE SITE TEST OF CONCRETE MATERIALS AND CONCRETE

Class of concrete :–
a In-situ, precast, plain and reinforced concrete work.
b Pavements (Apron and Hangar Floors.)
c Prestressed concrete for In-situ Main Beam.

MATERIALS	TYPE OF TEST	PLACE OF TEST	RECORDING OF TEST	FREQUENCY OF TEST	REMARKS
Cement.	Compressive strength of cement B.S. 12 –	Independent Authority.	Independent Authority Certificate.	One series of tests every 2 weeks.	Cement tests to be correlated with specific set of concrete cubes.
	Manufacturers Test.	Manufacturers Laboratory.	Manufacturers Certificate.	One series of tests for each consignment delivered.	
Fine Aggregate.	Sieve Analysis.	Site Laboratory.	As per B.S. 812 Appendix B.	Once every day.	Only when concreting is in progress.
	Determination of Organic Impurities.	ditto	On Form No. 3.	Once every week.	ditto
	Determination of Clay silt and fine dust.	ditto	ditto	ditto	ditto
Coarse Aggregate.	Sieve Analysis.	Site Laboratory.	As per B.S. 812 Appendix B.	Once every day.	Only when concreting is in progress.
Concrete.	Compacting factor test.	Site Laboratory.	Concrete control Record (Daily.)	At frequent intervals.	Only when concreting is in progress.
	Cement/Aggregate ratio (by weight.)	ditto	ditto	ditto	ditto
	Water / cement ratio by weight.	ditto	ditto	ditto	ditto
	Works Cube Tests.	ditto	ditto	One set of 3 cubes per 50 cubic yds. of concrete.	For concrete Class (a)
				One set of 3 cubes per 100 cubic yds. of concrete.	For concrete Class (b)
				One set of 3 cubes per lift per day.	For concrete Class (c)
	Density Test.	ditto	ditto	Per cube tested.	
	Cement Contest lbs./cubic yard.	ditto	ditto	At frequent intervals.	
Equipment.	1 Weigh-batching machine.	Check accuracy twice daily.			
	2 Water measuring device.				

Table 2.—Routine site test of concrete materials and concrete.

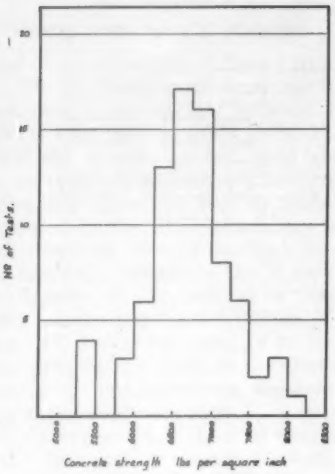

Table 3.—Pavement concrete.

the results of tests carried out on the pavement concrete and prestressed concrete are shown in Tables 3 and 4 respectively.

The following are details of the various mixes used and the degree of control obtained.

(1) Concrete for in-situ reinforced concrete construction

Design Strength	4,800 lb/sq. in. at 28 days
Minimum Strength	3,300 lb/sq. in. at 28 days
Cement/Aggregate ratio	1 to 6·5
Water-Cement ratio	0·59
Compacting Factor	0·90
Standard Deviation	572
Co-efficient of Variation	12%

(2) Concrete for in situ prestressed concrete main hangar door beam

Design Strength	7,500 lb/sq. in. at 28 days
Minimum Strength	6,000 lb/sq. in. at 28 days
Cement/Aggregate ratio	1 to 3·2
Water-Cement ratio	0·36
Compacting Factor	0·91
Standard Deviation	472
Co-efficient of Variation	6·25%

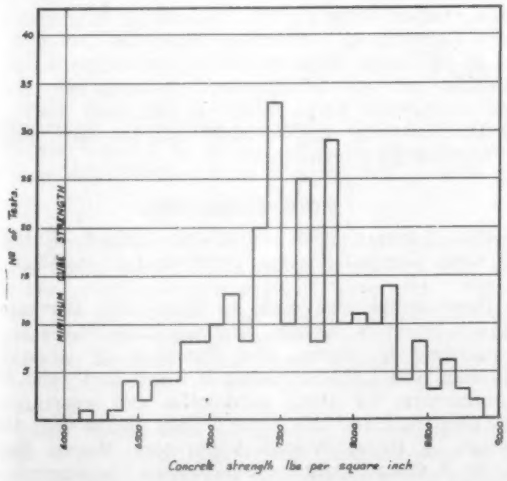

Table 4.—Prestressed concrete.

(3) Concrete for hangar and apron pavements

Design Strength	6,250 lb/sq. in. at 28 days
Minimum Strength	5,000 lb/sq. in. at 28 days
Cement/Aggregate ratio	1 to 6·3
Water-Cement ratio	0·482
Compacting Factor	0·807
Standard Deviation	535
Co-efficient of Variation	9%

10. Programme and Progress

A master programme was drawn up in conjunction with all sub-contractors in the early stages of the contract when sufficient details of construction and services were available. A series of *Star* dates was decided upon at key stages in the construction of the first hangar in order that any new development could be incorporated and reviewed relative to the final handover date of the whole project. No extension of time was found necessary and in the end the base was handed over to the clients three months ahead of the original target. Weekly progress charts were drawn up by the Planning Engineer initially with the section Foreman. The various trades were then extracted on to separate sheets (i.e. concrete, brickwork, reinforcement, excavation, etc.), and these were modified as required by the Planning Engineer to suit the mixer output, type of concrete mix, availability of plant and materials. Priority items were highlighted, or if any operation was out of sequence or falling behind the dates shown on the master programme. Fortnightly progress sheets were submitted by all sub-contractors to draw attention to impending trouble spots, delivery dates, difficulties, bottle necks, etc., before such items reached serious proportions.

11. Site Organisation

The Consulting Engineers had a Resident Engineer on the site, who was able to give prompt decisions and approvals to all matters arising in connection with the structural and civil engineering work. To assist him in his duties, he had on his staff an Assistant Resident Engineer, Mechanical and Electrical Engineers and two inspectors. The Quantity Surveyor had his staff on the site in charge of the Resident Quantity Surveyor.

The Main Contractor decided, due to the magnitude of the contract, to give complete control and running of the works to an independent and self-contained unit resident on the site, under the direction of a Project Manager. The make-up of this unit is shown in Table 5.

Four main meetings were established throughout the contract as follows :

(1) Planning and Progress Meeting held monthly and attended by senior representatives of Consultants and Contractors.

(2) Co-ordination Meetings held on site every two weeks and attended by representatives of Consultants, Main Contractor and Sub-Contractors.

(3) Resident Engineers' Routine Meetings held weekly and attended by Main Contractor and Sub-Contractors.

(4) Handover Meetings held as required prior to handover of a particular section of the Contract and attended by Resident Engineer, Specialist Resident Engineers, and Main Contractor.

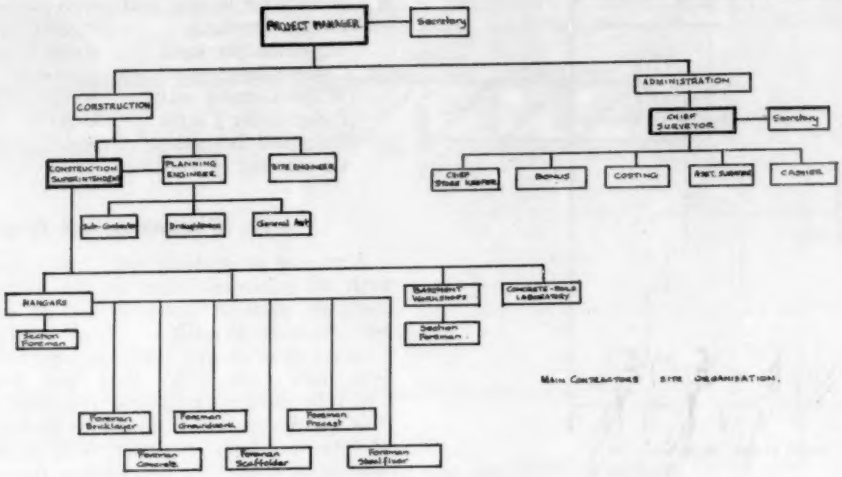

Table 5.—Main contractors site organisation.

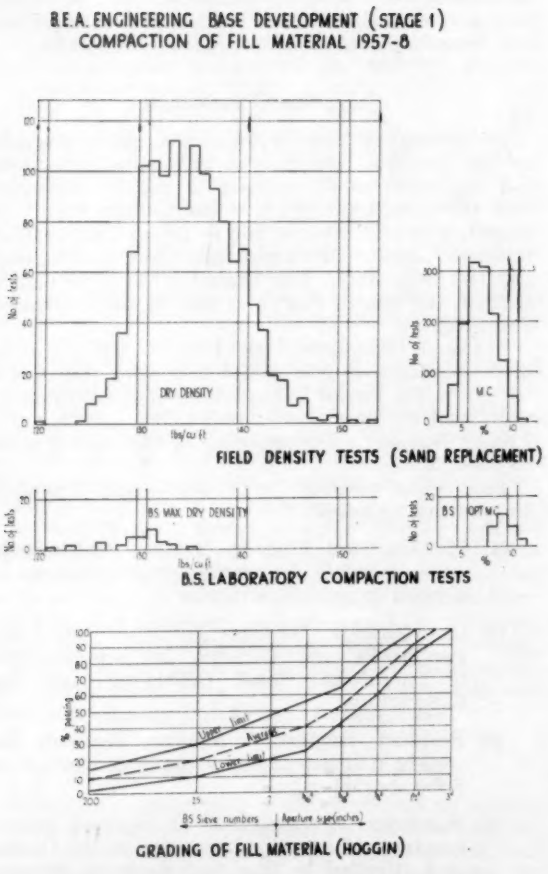

Table 6.—Grading of fill material (hoggin).

12. Conclusion

The value of the works described in this paper is of the order of 5·5 million pounds. The period over which the work was constructed was three and a half years. 15,630 tons of precast concrete was manufactured and transported to the site. Throughout the period of the Contract no fatal accidents or strikes occurred. These achievements could not have been accomplished were it not for the teamwork of the Consultants and Contractors, together with the many other interested parties concerned with the project.

From the designers' point of view, the detailing of precast concrete construction called for great care in making due allowance for tolerances, and furthermore a detailed knowledge of the facilities the Main Contractor would use for the construction. The success of precast concrete construction of the magnitude used in this project, rests on early joint collaboration between the Consulting Engineer and Main Contractor. A further important factor is the great advantage to be gained by producing, as far as possible, one set of drawings showing all services and fittings to be installed in the final work; to enable this to be accomplished calls for early issue of sub-contractors drawings.

Acknowledgements

The Authors wish to thank British European Airways Corporation for permission to publish this paper.

They would also wish to thank the Partners of Messrs. Scott & Wilson, Kirkpatrick & Partners, the Consulting Engineers, and Directors of Holland & Hannen and Cubitts (Great Britain) Ltd., the Main Contractors, for their permission and assistance in the preparation of this paper. They also wish to thank Messrs. J. Roger Preston & Partners, Messrs. Barlow Leslie & Coombs, for information on the services and all those who assisted them in the preparation of this paper.

APPENDIX I

Compaction of Fill Material

The fill material was hoggin, obtained from a number of sources. The grading varied, as shown by Table 6; the maximum size was generally between $1\frac{1}{2}$ and 3 in. with 2 to 15 per cent passing the No. 200 B.S. Sieve.

Histograms are shown for all field density test results and corresponding moisture contents, and of the maximum dry densities and optimum moisture contents obtained in the B.S. laboratory compaction test. Field densities were measured by the sand replacement method; owing to the stoney nature of the soil, the results of this test scatter fairly widely from the true value, and tend to average above the true average. On the other hand, densities obtained in the B.S. compaction test are reduced by the elimination of large particles from the test specimen. Taking these factors into account, it is apparent that the standard of compaction in the field was satisfactory. As expected, the best moisture content for field compaction was lower than the B.S. optimum.

In analysing the results, histograms were first compiled for each month, in an attempt to detect the influence of different gradings of material, methods of compaction or times of the year, but no correlation was found between these factors and the relative densities obtained. Laboratory compaction tests and gradings were carried out at frequent intervals.

The field density tests were carried out on the average at the rate of one per 100 sq. yds. of each layer. The method of recording results was in accordance with Form K, B.S. 1377—1948 with added columns to cover co-ordinates, type of compaction equipment and number of passes.

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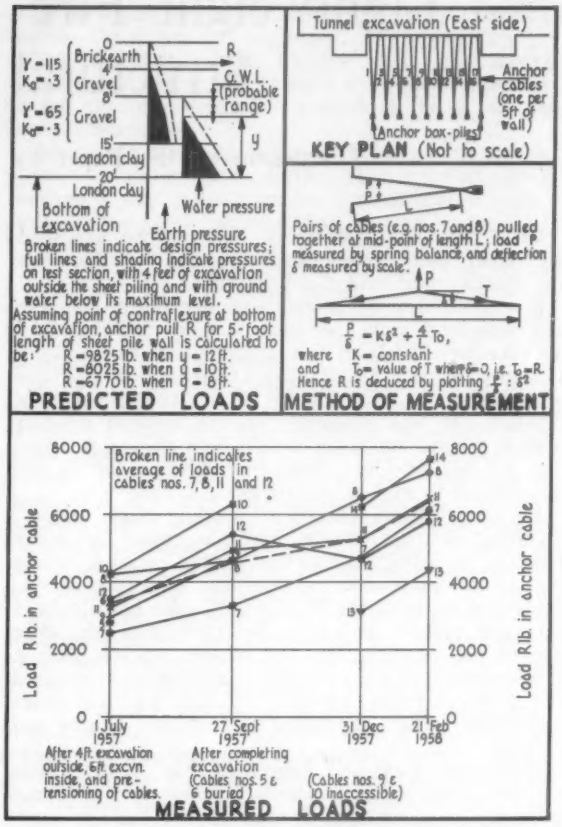
"Hangars at London Airport.—Design of Large Span Prestressed Concrete Beams," by A. J. Harris, B.Sc.(Eng.), A.M.I.C.E. *The Structural Engineer*, Vol. XXX No. 10, Oct. 1952.

"The Design and Construction of the British European Airways Hangars at London Airport, with particular reference to Prestressed Concrete," by D. H. New, B.Sc.(Eng.), M.I.C.E. *Proceedings of the Institution of Civil Engineers*, Vol. 2, April, 1953. Number 1.

APPENDIX II


Tunnel excavation (East side)
Anchor cables (one per 5ft of wall)
Anchor box-piles
KEY PLAN (Not to scale)
Brickearth
Gravel
Gravel
London clay
London clay
G.W.L. (probable range)
Bottom of excavation
Water pressure
Earth pressure
Broken lines indicate design pressures; full lines and shading indicate pressures on test section, with 4 feet of excavation outside the sheet piling and with ground water below its maximum level.
Assuming point of contraflexure at bottom of excavation, anchor pull R for 5-foot length of sheet pile wall is calculated to be:
R = 9825 lb. when y = 12 ft.
R = 8025 lb. when y = 10 ft.
R = 6770 lb. when y = 8 ft.
Pairs of cables (e.g. nos. 7 and 8) pulled together at mid-point of length L, load P measured by spring balance, and deflection δ measured by scale.
PREDICTED LOADS
METHOD OF MEASUREMENT
Broken line indicates average of loads in cables nos. 7, 8, 11 and 12
Load R lb. in anchor cable
1 July 1957
27 Sept 1957
31 Dec 1957
21 Feb 1958
After 4ft. excavation outside, 6ft. excvn. inside, and pre-tensioning of cables.
After completing excavation (Cables nos. 5 & 6 buried)
(Cables nos. 9 & 10 inaccessible)
MEASURED LOADS

Basement sheet-pile wall: Measurement of anchor loads

"The Planning and Design of Maintenance Bases for Civil Aircraft," by E. O. Measor, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E., L. B. Haley, M.Eng., A.M.I.C.E., and A. J. Harris, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. *Proceedings of the Institution of Civil Engineers*, Vol. 14, pp. 363-382, December, 1959.

Book Review

Elementary Structural Analysis, 2nd Edition, by C. H. Norris and J. B. Wilbur. (New York & London: McGraw-Hill, 1960). 9 in. × 6 in., 651 + xviii pp. 81s. 6d.

The general format and method of presentation of this book, which deals comprehensively with the basic theory of structural analysis, have been retained in this second edition, but since 1948 considerable progress has been made in various aspects of structural engineering so that it has been necessary to make major changes in the text to comply with these developments.

A foreword has been added which gives an historical review of structural engineering from its very early beginnings and a survey of the position at the present day. Another new chapter deals with the plastic behaviour of structures, and as a result of the increasing use of electronic computers, an important new chapter on matrix methods of structural analysis has been included. In line with recent rapid technological developments, a section on advanced structural mechanics has been added, giving an introduction to the theory of elasticity, and of plate and thin shell analysis and analysis of buckling behaviour.

Several chapters appearing in the first edition have been rewritten and amplified and the sections on miscellaneous structures such as aeroplanes, ship and chemical engineering and on gravity structures have been omitted from the second edition.

Many worked examples are included in the text and as before problems for solution are given at the end of the chapters and in this new edition answers have been supplied for approximately half of these.

Lightweight Fire Protection and the Structural Engineer *

Discussion on the Paper by A. R. Mackay, A.M.I.Struct.E.

Mr. A. W. HILL (Member) said that he had read the paper with considerable interest, but felt that Mr. Mackay had overstated his case.

For multi-storey construction reinforced concrete was considerably cheaper than encased steelwork, and the economy had been quoted as high as 35 per cent† as compared with Mr. Mackay's figures of 15 to 20 per cent, before the introduction of the 1959 revision of B.S.449. It was obviously in the general interest that structural steelwork should be more competitive, but Mr. Hill did not think the solution lay in the direction indicated in the paper.

In the Introduction Mr. Mackay had stated: "During the past decade there has grown up in the building industry a conviction that a concrete casing for the fire protection of structural steelwork is outmoded in multi-storey buildings." Mr. Hill doubted this and pointed out that fire protection was only one of the qualities required of the concrete casing. It also protected the steel from corrosion and increased the load carrying capacity of the member, both important factors especially for external beams and columns.

Commenting on the figures in Fig. 2, he agreed that they did not give a reliable guide to the overall economy of steel framed multi-storey buildings embodying lightweight fire protection. The figures given in Fig. 1 were based on B.S.449 : 1948, and he was sure that Mr. Mackay would agree that if the building was redesigned to the 1959 British Standard, a substantial economy would be shown. In the absence of specific data it was only possible to generalize, but Mr. C. Gray, in the discussion of a paper to the Institution of Civil Engineers† about a year ago, had advised us that the revision would provide an economy of 15 per cent in the cost of concrete encased structures due to the 5 per cent increase in the allowable stresses in the steel and the increased use of the structural value of the concrete encasement. If it was assumed that of this 15 per cent economy, 10 per cent was due to the increase in the structural value of the encasing concrete, the figure for Scheme 1 would be 90 and thus less than for Scheme 2.

In Fig. 3 Mr. Mackay again used figures based on B.S.449 : 1948 so that for a true comparison the value in column 2 should be reduced to 90. Column 3, therefore, suggested that by using a concrete encasement for the columns and lightweight protection for the beams, an economy of 5 per cent was possible as compared with all concrete encasement.

This suggestion was, perhaps, worth further study in regard to varying spans and loadings and also in relation to the latest provisions of B.S.449, since Mr. Hill did not think it would be of general application.

Mr. Mackay had referred to the danger of hollow casings in the case of fire when they were not sealed at each floor level. He had not mentioned the danger of corrosion if condensation occurred on the steelwork. Did he think this was a serious difficulty?

Any attempt to compare the costs of work in America with those of similar work in this country was likely to be misleading. While Mr. Hill accepted the extracts as quoted, it was possible to quote examples from America which showed that reinforced concrete was cheaper.

Mr. MACKAY said he would prefer to give a detailed reply to Mr. Hill in writing.

Mr. C. S. GRAY (Associate-Member), said it seemed that in some cases where designs were in accordance with B.S.449 : 1959 there was no advantage in encasing beams in concrete. The lightweight encasement did not decrease the carrying capacity of the beam which the meeting had been discussing, the normal floor beam in the multi-storey building.

Turning to Fig. 3, he said it was true that the figures were calculated on B.S.449 : 1948. If, on the other hand, one looked at scheme 1 and re-designed it according to today's requirements, from his own investigations on a 10-storey building the saving of steel weight was between 10 and 12 per cent. The cost saving was more than that because the volume of concrete in the lower stanchion casings had been reduced.

To be completely effective, the lightweight encasement should be used for floors which did not require shuttering.

There was a further point. At the top of page 23 Mr. Mackay states that "..... the main structural asset of a concrete casing, monolithic with the structural floor, loses its value almost completely." It should be emphasised that Mr. Mackay meant the beam casing, not the concrete casing of the stanchions.

Mr. Gray did not subscribe to the view that this was not a noteworthy advance. To his mind it was one of the things which would put structural steel back on the map and, as he had said to the Institution of Civil Engineers, the effect of the new B.S.449 would be a saving of 10-15 per cent; to this we could add Mr. Mackay's 8½ per cent, not take it away, as had been suggested by a previous speaker.

Mr. J. R. LOWE (Member) congratulating the Author on his fascinating historical notes, said it was not often that engineers said anything about the history of their subject.

He agreed with most of the points that had been made, but, like Mr. Hill, he did not believe that comparisons between steel and reinforced concrete buildings in the U.S.A. were valid for this country. It was his experience that the cost of a well designed structure, making use of up to date methods, of either steel or concrete construction, was about equal.

One consideration which had some bearing was the period of fire protection required. For instance in schools, only half an hour was generally required, but it was frequently impracticable to provide only this

* *Read before the Institution of Structural Engineers at* 11, *Upper Belgrave Street, London, S.W.*1, *on the* 21*st January*, 1960. *Mr. Lewis E. Kent, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E. (President) in the Chair. Published in "The Structural Engineer," Vol. XXXVIII, No.* 1, *pp.* 20-25.

† Creasy, L. R. "The Economics of Framed Structures." Proc. Inst.C.E. Vol. 14, Dec. 1959.

amount in a steel structure; whereas almost any reinforced concrete structure was inherently good for half an hour. In these circumstances the use of concrete should be more advantageous than in buildings requiring longer periods.

On the other hand, when a suspended ceiling was used the extra cost of making it fireproof was very small. Nearly all office, school and factory buildings were in fact so treated.

One criticism that he offered concerned the use of plaster. The present trend, which he believed was increasing, was to do away with site plastering as far as possible due to difficulties with that particular trade.

Another point, of importance in schools or public buildings, was the durability of plaster. He had been informed by architects that in many instances plaster in schools was defaced after six months of use. The corners of columns were chipped off and surfaces had frequently been deliberately scratched with knives.

Mr. LOWE mentioned a method which had been used in Germany for treating circular columns with gypsum plaster. A complete storey height was poured in one lift, inside an aluminium shutter. The finished surface was very good and the shape was inherently less liable to damage than was a square column.

One way of avoiding " wet " trades on site was the use of prefabricated sections of asbestos, or similar materials. He asked for the Author's comments on this method.

He asked the Author whether he could give approximate costs per foot run for providing a half hour resistance for a column; prices he had obtained varied considerably. The cheapest price he had received was for a proprietary type of fire resisting plaster finished with a skim coat of cement sand mortar, in respect of which he had a quotation of 10/- per lineal foot for a 7½ in. square box column. Another quotation related to a type of asbestos sheathing and the price quoted was 4/6 - 5/- per foot run. He doubted this price, which seemed extremely low, and asked whether confirmation of it was available.

There were certain cases where an engineer did not need to provide any special protection, i.e., where beams and columns were erected within the thickness of a brick wall. The easiest way of providing fire protection in that case was to surround the member solidly with bats and mortar. With the addition of plaster to the wall the protection provided was the cheapest and perhaps the best possible.

Mr. MACKAY, dealing with school buildings, agreed that usually the fire protection required was for only half an hour. That could be achieved by using plaster and metal lath; and in circumstances where the casing of a beam would ordinarily be plastered, one would not be using any more plaster than would normally be used for decoration over the concrete casing.

Concerning columns, he said he was generalizing, but he personally would advocate casing them with reinforced concrete in the manner recommended in B.S. 449 : 1959; one would use the right fire protection material in the right place with greater advantage.

He could not recall the cost per lineal foot of, say, a 15 in. x 9 in. casing with vermiculite and plaster or with plaster alone, and if he were quoted figures he would treat them with reserve. Using metal lath and formers, such as he had illustrated, on a beam 16 in. x 6 in. would cost approx. 3/9d. per sq. ft., and the plaster itself was working out at 1/6½d. per sq. ft.

Mr. Mackay explained that he does not advocate one particular method of fire protection to the exclusion of every other. The beam casings in a staircase might very well be done with concrete for example; these serve the purpose of supporting the brickwork as well as the floors and landings.

Mr. Mackay doubted whether bats and mortar would be expected to withstand a two-hour fire test; he felt that one could find something better, at any rate for beams.

Columns have been cased with brickwork for fire protection quite successfully. The inherent disadvantage is that most often the overall size of such a casing is too great; greater than it need be by about 5 in. on each face.

He felt that the rate quoted for the 7½ in. square box column with a plaster finish—namely 10/- per lineal foot—was reasonably correct, but he did not know of an asbestos sheathing system as low as 5/- per lineal foot of such a column.

Mr. P. A. DENISON suggested that the use of a completely dry casing had been somewhat overlooked and that it was in the development of a dry casing that the greatest economy for structural steel would be gained. He felt that the latest test results and development of a new type of asbestos board casing had probably not been available to the Author, but he thought it would be of interest to the meeting to know that a new type of asbestos casing was now available that could be screwed into itself.

This new method superseded the previous method of using an asbestos board, whereby a thin (maximum ¼ in.) sheet was screwed into asbestos battens. This previous method had not proved economical.

With the new method, ¾ in. asbestos board casings in 10 ft. lengths were screwed into each other to provide perhaps the simplest possible way of devising a casing. A two-hour fire resistance grading had been gained by this method in a recent J.F.R.O. test.

The previous speaker had indicated that with concrete and also with vermiculite, it was not really possible to take advantage (so far as the encasement was concerned) of any lesser fire resistance requirements. With the asbestos casing, the two hour fire requirement was met with the use of ¾ in. asbestos board on four sides. With lesser requirements, maximum economy could probably be gained by reducing the thickness of board on two sides. There was certainly scope for further testing here, in order to explore the most economical use of the correct thickness that should be used.

Mr. Denison said he had examined in some detail the application of this new method to the protection of ' Universal ' sections and it was clearly possible to produce a standard range of casings in 10 ft. lengths, limited perhaps to as few as six different standard widths. A simple method of location had been devised whereby this range of casings could be applied to the whole range of ' Universal ' sections. Thus the introduction of ' Universal ' sections had greatly simplified the use of a dry casing system.

(A later speaker claimed that it was not possible to standardize on a limited range of casing widths, owing to the fact that the strength of any given ' Universal ' section was increased by thickening the flange. Mr. Denison did not have the opportunity to reply to this point, but would like to add the note in this draft that there is in fact, no difficulty in such standardization. A given ' Universal ' section is strengthened by thickening the web, which does not affect the outer girth, and also by thickening the flange. This latter dimension affects the casing dimension in one direction only. Thus one dimension for a given section can be standardized and so long as the other side caters for the maximum thickness of flange, all variations for

that section within the maximum, can be encased accordingly.

A simple locating bracket screwed to two sides of the casing, ensures that the casing is firmly located. It is, therefore, possible to reduce the number of external variations in casing widths, to approximately six dimensions. The Company concerned with this development is producing a chart showing the correct casing to choose, according to the dimension of the stanchion).

Mr. Denison went on to say that the development of this dry casing method perhaps enabled the encasement of steel to proceed on rather more sophisticated lines. It is now possible, for instance, to produce these casing sides with applied finishes, so that the fire protection encasement and column finish could be dealt with in one operation.

Mr. Denison felt that the cost analysis of these new methods was at this stage a difficulty, as it was obviously very hard to get at the precise cost advantages again by these more sophisticated methods. The previous speaker had mentioned the development of the dry asbestos casings, indicating a very low foot run price. He felt that this information was wrong and that for a 10 in. × 8 in. stanchion, the foot run cost was in the region of 16/- per foot run for the finished casing (ready for a paint finish).

It was not quite fair, however, to compare one foot run cost with another. He felt for instance, that maximum advantage of this method was gained when the floor design was related to its use. Use of a suspended fire resistant ceiling to protect the beams would mean that where the stanchion passed through a fire resistant ceiling, the stanchion would not require fire protection above that point. The associated use of the ceiling in this manner contributed to the economy of the casing.

Again, it was probably difficult to assess the precise cost advantage of using a method of encasing steel which incorporated a finish, at the same time as providing the fire protection. Perhaps it was possible to go even further, and in some circumstances, to consider the use of the column casing as a service duct. It would certainly be possible to use part of one of the sides as an access panel. Here again, it was difficult at this stage to determine the overall cost benefit.

Mr. Denison concluded by hoping that use of alternative dry casing methods would not be overlooked, and he felt that there was a great deal of scope for development in this direction.

Mr. FREDERICK S. SNOW, C.B.E. (Past President), remarked that a good engineer is able to choose and use the materials he requires in a building to the most economic advantage.

He had noted the fact that a prospective purchaser generally preferred a steel frame building with encasements. Somehow steel always caught the imagination.

It was quite obvious if we looked at the casing (examples of its use were exhibited at the meeting) and took into consideration the weight of Vermiculite, which was approximately 30 lb/cu. ft. against concrete at about 144 lb/cu. ft. there must be a considerable saving in weight and cost. Using a normal concrete encasement the foundations would be of a size commensurate with the weight of the structure, whereas a considerably taller building could be erected by utilizing Vermiculite cladding, as described in Mr. Mackay's paper.

As we all know structural steelwork had suffered from the competition of reinforced concrete over quite a number of years; this was due, he thought, to "sitting back" when there was an easy market. Now conditions had changed and in his opinion, we must use more structural steelwork, thereby using more coal to the benefit of the coal mining industry.

Vermiculite was a Commonwealth product and had been largely developed in this country by the Rio Tinto Company and it had helped our Commonwealth trade.

If one considered the erection of a reinforced concrete structure, we first cast the foundation, then the columns in some form of steel or timber shuttering, proceeding upwards to the roof. On reaching the roof or some intermediate level, we proceeded to complete by plastering, etc. With the method discussed by Mr. Mackay, we started off with smaller foundations and built up the structural steel with universal sections to the top, putting in the floors as the height increased, which could be precast or prestressed.

The method of design was unique and had not been used anywhere in the world in its present form. The technique was entirely new and it reduced cost. Obviously it must be cheaper to work in this way because the work of the various trades could be carried out as the height of the building increased and the men would be able to work in the dry. It would be an extremely interesting exercise if somebody would erect a really good reinforced concrete building and one of this design side by side.

Finally, he pointed out that in the construction industry both in this country and abroad, there was a place for every type of structure and in no case would the use of one preclude the use of the other.

Mr. F. W. SLATTER (Member), said reference had been made in the discussion to the new universal beam, and it was made clear in the paper that the Author's researches were carried out on the basis of the old British Standard 449 before universal beams had really got into their stride.

Possibly further economies could be made in the costs given in the paper by utilising the proposed new system and also B.S.449 : 1959.

Mr. MACKAY replied that in Scheme 3 he had made use of the progressive practices recommended in the new B.S.449 but did not make use of universal beams. Where he had used compound sections in Schemes 1 and 3 he had also used compounds in Scheme 2.

A point which was very significant but had been overlooked in the discussion so far, was that one quarter to one third of all the concrete that went with the steel frame was used, by and large, for fire protection. The beam casing was carried by the steel itself, and it was surely absurd that as much as one third of all the concrete on a steel frame was used in such an unproductive way.

Mr. L. R. CREASY (Member), asked if Mr. Mackay would kindly confirm the prices he had quoted to a previous speaker in connection with the casing of a 16 x 6 rolled steel joist. These, he believed, were 3/9 per sq. ft. for the mesh and laths and 1/6½ per sq. ft. for the plaster. He supposed that it must be applied to the periphery of the casing. Allowing for a 2 in. cover these rates appeared to be equivalent to approximately 18/- per foot run for the casing of the 16 in. joist complete. Where the cost of fabricated steelwork was £70 per ton the cost of the joist was approximately 30/- per foot run. The cost of the cased beam was about 48/- per foot of which something of the order of one-third was spent in the casing. This was a proportion which one could expect from normal concrete casings and it was disappointing to find that vermiculite did not indicate a more marked advantage.

This factor of one-third was also a measure of the difference in cost between a cased steel and reinforced concrete frame to which reference had been already made. In the latter case the floor slab was a working member of the frame and a comparison of these alternatives must include in each case the cost of the columns, beams and foundations together with the floor slabs and casings.

An analysis on this basis showed, generally, a cost margin of between one-quarter and one-third of the steel frame with concrete casings in favour of reinforced concrete. This gap was appreciable and must be attacked from many sides if parity was to be achieved.

From an independent investigation which had been made of the example indicated in the paper, it appeared that the substitution of vermiculite for concrete casings to the beams makes a reduction of not more than 10 per cent in the overall cost of the cased steel frame including the floor slabs and foundations. This was most valuable but was insufficient by itself to close the gap between the cost of cased steel and reinforced concrete.

A further important advance would be made when our normal designs allowed for the contribution to the strength of the frame which was made by the composite action between steel beam and floor slab. An additional improvement of the order of 10 per cent could be achieved by this means and this then brought parity within striking distance.

Mr. MACKAY thanked Mr. Creasey for, and welcomed his reference to the advantage of composite action; composite construction was a very important step which we should have to take sooner or later. Whatever interaction there might be between the beams and the slab, if it contributed something to a saving of cost it would be welcome. Nevertheless, the fact remained that the beams would still have to be fire protected and it was clear that the application of lightweight fire protection was an advantage. The two were not rivals, but were means to the same end.

(In a further reply the Author has written that the rates quoted of 3/9d. and 1/6½d. respectively were substantially correct. A corresponding concrete casing would appear to cost approximately 21/- per foot compared with Mr. Creasey's figure of 18/- for the plaster casing. However, in order to make the comparison really valid the cost of the finishing with plaster should be added to the concrete casing. The reason being that the two are only ready for comparison when they are each ready for decoration.

Render and set would cost approximately 10d. per sq. ft. of finished casing which adds a further 3/- per lineal foot to the concrete casing price.

In short the plastered concrete casing amounts to 24/- per foot compared with 18/- per ft. for the vermiculite gypsum plaster casing.

The Author feels that Mr. Creasey's independent investigation of the cost of a cased steel frame including floor slabs which showed a saving of 10 per cent is not incompatible with the above figures nor with those quoted in the paper which generally excluded floor slabs except that they included the cost of the extra floor shuttering necessary with hollow casings.

Since January, the Author has been concerned with a seven storey steel framed office building near London. It exploits B.S.449 : 1959 to the full, universal beams and reinforced concrete or vermiculite plaster casings as appropriate.

Preliminary estimates show that this building complete with floors and ready for services and finishes will cost almost exactly the same as a reinforced concrete framed building would cost on the same site. The Author hopes to submit an article or paper on this building as soon as he is free to do so).

Mr. G. B. GODFREY (Associate-Member of Council), observed that the large attendance that evening signified both the widespread interest being displayed in the fire protection of structural steelwork and the comparative dearth of information published on the subject.

If he could make any criticism of Mr. Mackay's paper, it was that it contained insufficient information. There were no illustrations. There was no bibliography, although the Author had given a fascinating historical introduction and had referred to specific American research. Could he ensure that these omissions were made good in his written reply to the discussion?

The title of the paper was " Lightweight Fire Protection and the Structural Engineer " but, in fact, it dealt almost exclusively with vermiculite-gypsum plasters. While members knew that vermiculite, in one form or another, had become extremely popular in North America and in Europe, they would also be aware that there were further methods of protecting structural steelwork, in addition to concrete encasement. No doubt, they would welcome information on the relative efficiency of the various methods.

During the last six or seven years the German Constructional Steelwork Association had sponsored extensive research into the problem of fire protection. The work had been described by Dr.-Ing. Paul Boué in a book entitled " Der Feuerschutz im Stahlhochbau, insbesondere von Stahlstützen " (" The Fire Protection of Structural Steelwork, especially Stanchions "), published in Cologne in 1959 by Stahlbau-Verlag, a copy of which Mr. Godfrey had presented to the Institution Library.

Having demonstrated with a lantern slide that the procedure for fire tests was virtually the same in the U.S.A., Germany, Sweden and Great Britain, among other countries, he stated that there was no valid reason why the German results should not be accepted here. Indeed, he could testify that in a recent fire test with which he had been associated, the result confirmed similar tests made in Germany. Mr. Godfrey then displayed Fig. 1, which had been adapted from Fig. 140 in the above-mentioned book.

The thicknesses of protection quoted were equally applicable to I-sections or tubes. A previous speaker had suggested that a range of precast casings should be produced for the more commonly employed Universal columns. It was an unfortunate fact, however, that a single precast element would not suffice for each serial size as the rolls were opened *outwards* to produce the various nominal sizes. In this respect, tubes were advantageous because the wall thickness was increased by reducing the bore, the outside diameter being constant. The same method was applied to rectangular hollow sections.

Mr. Godfrey stated that in 1958 both the President and he had had the opportunity of visiting the Palazzo Sidercomit, a seven-storey building then being constructed in Milan, the columns for which were tubes. There, semi-circular precast vermiculite elements, 3 cm. in thickness, had been clipped around the tubes.

Mr. J. R. Lowe had already mentioned the method of protecting the tubular columns in the twenty-four storey Mannesmann Hochhaus in Düsseldorf. In the more recently completed twenty-five storey building for Phoenix-Rheinrohr in the same city the tubular columns were protected by spraying 3 cm. of vermiculite plaster on to an expanded metal cage.

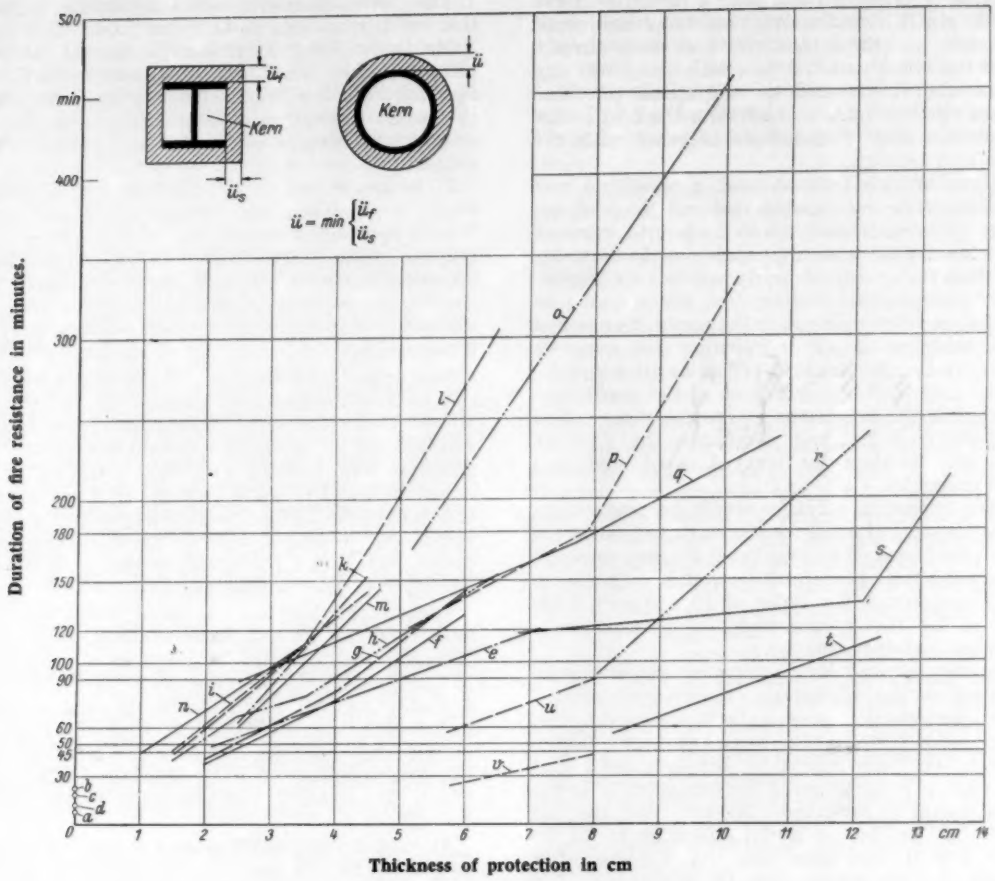


Fig. 1.

KEY TO FIGURE 1.

a	=	bare steel stanchion	k	=	vermiculite-gypsum plaster
b	=	bare steel stanchion with fire protecting paint	l	=	vermiculite-cement plaster
			m	=	vermiculite-cement slabs
c	=	bare steel stanchion with internal filling (rolled sections)	n	=	sprayed asbestos plaster (Limpet)
			o	=	solid concrete encasement (other than gravel-concrete)
d	=	bare steel stanchion with internal filling (tubes)	p	=	bricks in natural lightweight aggregates, with and without filling (Kern)
e	=	gypsum slabs	q	=	solid gravel-concrete encasement
f	=	lime-cement plaster	r	=	solid tiles
g	=	gypsum-sand plaster	s	=	perforated tiles with filling (Kern)
h	=	cement-bound mineral fibre mixture (Efawit)	t	=	perforated tiles without filling
			u	=	hollow tiles with filling
i	=	perlite-gypsum or perlite-anhydrite plaster	v	=	hollow tiles without filling

In conclusion, he felt that these methods were more applicable to tubes than plastering, although plastering would be an excellent solution for square or rectangular hollow sections.

Mr. L. A. ASHTON said the paper appeared to him to cover adequately all the points that needed to be covered ; and although he was aware that when dealing with fire protection it was easy to fall into errors, he could not find any fault at all with the paper. He offered his congratulations to Mr. Mackay.

Dealing, however, with one or two points raised in the discussion which appeared to be of major interest, he said that to refer to " most fires being caused by explosions " was rather an exaggeration. All the major fire losses in industrial buildings in recent years had been caused in other ways.

There was also the point about expanded metal in the encasement and whether it was acceptable because of the high temperatures to which it would be subjected. We had to bear in mind that in this use the metal was

not structural; it served the purpose of supporting the insulating material and it fulfilled this function even though it was exposed for a long time to high temperatures.

One speaker suggested that heat could be conducted through metal fixings to the structural members, but the path of heat conduction was so small, in comparison with the size of the members, that it was not a significant factor in raising their temperature.

On a broader issue, he said it was important to bear in mind that fire protection requirements were based on ensuring that a structure would give safety during a fire and would not collapse after a fire. There was another aspect, however, which had received rather little attention in comparison with behaviour during a fire and this related to fitness for use after a fire. Observations made during and after actual fires had shown that protective coatings of lightweight plaster were of very great advantage in protecting structures against damage, especially those of concrete, so that the prospects of reinstatement would be better than when there was no protection.

Mr. C. S. GRAY, referring to the book " Der Feuerschutz im Stahlhochbau " by Dr. Boué, which Mr. Godfrey had correctly described as a thesis which was written for the purpose of obtaining a Doctorate in Germany, said that as a document on research it was excellent; it covered English, American, Dutch, Scandinavian and Austrian researches. A drawback to that book was that it had 180 very closely written pages in German. Mr. Gray said he could supply translations of the list of contents, tables and diagrams.

Another book well worth looking at was " Rohbaufertiger Stahlskelettbau " by Dr. Bongarp, which set out examples of modern steel frame construction with fire protection by lightweight encasement. There were descriptions in German and very clear drawings.

Mr. MACKAY said he had no information from those sources, but he would like to have it.

Mr. WALTER GOODESMITH (Associate Member), referring to some research he had carried out on the subject of the development of fire protection of structural steelwork, mentioned that over the years there had been some 14 official and semi-official documents which had either guided or governed the use of materials for the fire protection of structural iron or steelwork in buildings, in this country.

It was not until the 1955 issue of the F.O.C.—F.P.A. Fire Grading of Elements of Structure used in Buildings that vermiculite plaster, (in addition to vermiculite cement) was mentioned for the first time, in the hollow construction section. These references to vermiculite plaster were based upon the work carried out by Mr. Ashton at the J.F.R.O. Fire Research Station at Elstree, and his tests had also covered most of the sections which Mr. Godfrey had shown on the slides at the meeting.

Mr. PETER GARDINER (Delegate Member of Council), said it was stated in the paper that casings must be sealed at junctions and at the seams. He asked if the sealing of the stanchions below the level of the beams and the seals at the angles of the walls were such as to provide fire protection.

Mr. MACKAY replied that gaps in stanchions could be sealed by the floor slab. Junctions, joints or corners in buildings were always sources of possible weakness, and sound building practice was as necessary as always. It would become good practice to produce a vermiculite plaster coating on the under surface of the slab, in effect a continuous membrane of fire protective material between the structure and the source of a potential fire.

Mr. H. SAUNDERS representing the company which produces vermiculite and the company which is responsible for marketing it, expressed appreciation of the privilege of being invited to this most stimulating discussion, and thanked the Institution for having arranged it.

It was because of Mr. Snow's interest that the study group had been formed, Mr. Mackay had been made available, and the invaluable work which led up to his paper had been done.

This was an example of the co-operation that existed between the various branches of the engineering profession. People in the mining world greatly valued the tremendous help they always received from other branches of the profession in dealing with their problems which were in the course of exploration and development of minerals. From his knowledge of the industry which was producing, processing and selling, vermiculite, all engaged in it would be ready to give structural engineers every backing as they progressed in dealing with the problems which, it was still evident from the discussion, were yet to be overcome.

Mr. L. A. FIRMINGER (Associate-Member) said it seemed to him that a system based on concrete encasement of the columns, taking full advantage of the increased permissible load in the cased columns and at the same time stressing the beams to $10\frac{1}{2}$ tons per square inch would enable a steel structural frame to compete with reinforced concrete. The difficulty was the deflection in the beams when fully stressed.

In nearly every case he had found that with a 70 lb/sq. ft. superimposed load the beams could not be anything like fully stressed if simply supported due to the deflection limits.

Somehow, by welding or by some other system, we must develop a means of fixing beams to stanchions or embodying false continuity so that advantage could be taken of the full permissible working stress in beams.

A saving of 12 per cent in a steel frame designed in accordance with the latest B.S.449 as against that of 1948 was quoted by Mr. Gray.

Mr. Firminger asked if this were on the basis of a continuous structure or one with simply supported beams and if Mr. Gray would care to comment on this problem.

Mr. MACKAY replied that the problem raised by Mr. Firminger was important and interesting, but in the particular research exercise referred to in the paper it was regarded as irrelevant.

Amendment No. 1 to B.S.449 : 1959, published 29th July, 1960, includes recommendations which allow for up to 10 per cent of the free moment in a beam to be regarded as a restraining moment at beam to column connections. An important proviso is that the column be encased with concrete in accordance with Clause 30b of the new B.S.

Mr. M. M. KHAN (Member), said that, in his experience, the cost of reinforced concrete beams of comparable depths was a little lower than the cost of plain steel beams without any casing. As regards reinforced concrete columns, they were a great deal cheaper than plain steel stanchions, again without casings.

He added that circular section reinforced concrete columns could be constructed which would cost no more than steel stanchions without casings.

Mr. H. P. HOLT (Associate-Member), said it was difficult to get a true comparison cost between concrete and lightweight cladding. The only way to do this was to go to tender.

Mr. Holt said he was at present engaged on the extension of a large building in the City of London

and the question had arisen as to whether it was better to encase the beams and stanchions in concrete or to clad them with lightweight material. Tenders were invited for cladding in concrete and cladding in vermiculite, to give one hour's protection against fire. The prices had shown a slight advantage towards the vermiculite cladding and therefore it was decided to adopt it. Apart from the cost it is also much easier to handle, which is an important point in a congested area.

Having adopted it, unfortunately those engaged in the work did not know enough about it. Mr. Holt and many others had come to the meeting to try to find out some more about it, but unfortunately, Mr. Mackay had talked only about the vermiculite/gypsum plaster and had not touched on other materials such as precast vermiculite slabs.

Mr. Holt thought that a much more efficient and quicker way would be to use precast cladding. There were one or two firms offering it in London.

Regarding vermiculite/gypsum plaster, it was noted that it involves the use of metal angles near the edge of the cladding and Mr. Holt was concerned whether the London County Council would not reject this, for the reason that steel was too near the surface. Therefore he asked if it had become apparent that to have metal too close to the surface had any detrimental effect or that it would be better that the metal should not be there at all.

Mr. MACKAY replied that the precast vermiculite and cement slabs, or vermiculite and gypsum slabs were available on the market from firms quite conversant with the material. His firm had specified them in a very large contract and they had been completely satisfactory.

The reason for concentrating on vermiculite gypsum plaster was simply because he found from preliminary research that it was the cheapest lightweight fire protection available.

The precast slab appeared to have more to commend it in installations where a plaster skim coat was not required as a basis for decoration.

Mr. L. R. KNOTT (Associate-Member), raised the point that in industrial buildings a great many fires were started by explosions and asked how the encasement performed under blast or shock conditions.

Mr. C. WRIGHT (Member), made the point that without the co-operation of the plaster manufacturers we should make little progress with plaster encasements. He agreed that the system discussed by Mr. Mackay was a very good one.

Next, he recalled that in 1934 he had designed the first vermiculite exfoliating stack used in Britain. The Company he served at that time introduced vermiculite plaster to the building trade for insulating the soffite of concrete roof slabs.

There was great difficulty pleasing the plasterers, who were not inclined to work at the quick tempo of their U.S.A. counterparts. To please them the setting time was extended. Thereafter there were many complaints, especially during bad weather conditions, which cost the manufacturers much expense for loss of labour and replacement of material.

It was finally decided that it was necessary to emulate the conditions provided in the U.S.A. where buildings were enclosed and heated if perfection was to be obtained.

Where this procedure was followed, the application of this particular plaster was quite successful. Comparisons of cost with U.S.A. practice should take into account many differences. U.S.A. gypsum was 95 per cent pure. In this country gypsum generally was of poor quality. The selling price of gypsum plaster in Britain was very low (compare it with coal) and was made possible by large sales of plaster board. If the ratio was disturbed by an increase in demand the selling price must, of necessity, be reconsidered, which made it imperative that co-operation of the manufacturers be obtained.

Mr. MACKAY, commenting on Mr. Wright's remarks, said it was not enough to have a good idea ; it was necessary also to have that idea at the right time. It appeared that there had been some premature disappointments—he would not say "failures"—in using vermiculite/gypsum plasters ; but these things had in fact been taken care of. There had been conferences with the National Federation of Plastering Contractors and they had endorsed the use of vermiculite plasters for a number of years. The most important safeguard was that the proportions of vermiculite and gypsum be properly controlled ; another was that application should be by conscientious workmen. The plasterers themselves were already quite familiar with the vermiculite/gypsum plasters. Those plasters had been used for years as the natural backing coat for plaster in ordinary reinforced concrete work, and the plasterers knew how to handle them satisfactorily.

There were some examples of the lightweight fire protection technique on display at the meeting, from which Mr. Wright would see that the question of protecting the components against corrosion had been given careful thought.

THE PRESIDENT, at the conclusion of the discussion, which he described as stimulating, said he was a little surprised that there was no mention in the paper of pre-fabricated lightweight casings ; but one now knew the reason why. There might be something in this point, and perhaps one thing in their favour could be that they would give a straight line on the side of a beam, which a man who was placing the material with a trowel might not achieve.

Conveying the meeting's approbation to Mr. Mackay for the way in which he had replied to the questions, the President said he had rendered quite a service in putting up this particular subject for discussion, and he had done very well. Obviously it was a subject to which we had got to give attention, and this was a very good starting off point.

Written Discussion

Mr. E. J. BUCKLEY (Member), commented that Mr. Mackay's paper had been useful in drawing attention to the potentialities of lightweight fire protection to steel frames. The value of the paper was however, restricted by the concentration of detailed consideration to only vermiculite gypsum plaster (wet construction). There were of course, various forms of " dry " cladding (some proprietary) which would appear, at least in some circumstances (and when correctly designed and used) to have considerable advantages over the vermiculite gypsum plaster as suggested by Mr. Mackay, and any information that could be given on the various alternative lightweight claddings would be useful. The various influences that affect the economies to be gained by lightweight claddings were complex, and were probably not amenable to the simple cost analysis given in the paper as, in any investigation of this nature, some assessment must be made of the economic effect of the construction time factor. It was evident that with the very high level of financial liability involved both in site acquisition and essential preliminary charges any reduction in construction time, giving an earlier return on invested capital could, of itself, be of overriding importance. In this context

also it was possible that the advantages of speed of construction of lightweight cladding stanchions could more than offset the more obvious economies gained from the use of solid loadbearing concrete encasures. Further, by careful design and with lightweight cladding, a reduction in the overall finished dimensions of stanchions could often be achieved.

The possibility of the corrosion of the steelwork did not appear to have been covered in the paper and it would be interesting to know if this did, in fact, occur under normal, or perhaps special conditions, and what preventive measures, if any, are recommended.

In tall buildings it was becoming increasingly common to rely upon the floor construction to transmit lateral forces to special sway bents or transverse walls. It might be necessary, with the omission of the concrete beam casings, to take some special measures to ensure the efficient transference of the loads from the frame to the floor construction.

The AUTHOR replied that the question of corrosion had been dealt with to some extent in his response to Mr. Hill. There is substantial evidence from existing buildings of long standing that anti-corrosion treatment is unnecessary in a building wherein there is no abnormal corrosive condition. In a building such as a laundry, or parts of a building such as a hotel kitchen, or in many factories the corrosive attack is recognisable and should be dealt with on its merits.

Apart from external beams and columns, the Author is convinced that the risk of corrosion will only invalidate plaster fire protection to a very minor degree over the whole field of structural steel frameworks.

In the Author's opinion the omission of a concrete casing in favour of lightweight casings will not upset the equilibrium of structural frameworks, where in addition to its normal junction the floor slab transmits lateral forces to vertical wind girders of one form or another.

The effect of omitting the concrete casing should not be overlooked of course but it will be found that if some additional lateral 'key' is required, it can be provided by other means, more economically than with a concrete beam encasement.

Mr. A. W. HILL in a written communication stated that in order to clarify the report of American practice given in the paper, which appeared to have been abstracted from the December 1958 issue of "Progressive Architecture," he had obtained the following information from the United States :—

(a) All the examples quoted in Figure 5 are cost estimates which have not been confirmed by actual tenders.

(b) The data given for the Illinois project referred to the first application for tenders. Only two tenders were received, one for steel and one for concrete as quoted in the paper, but as these were both so much higher than anticipated the job was withdrawn by the General Service Administration and redesigned. The contract was awarded in February 1960 in reinforced concrete. It is not possible to determine precisely why the original concrete figure was higher than steel, nor why it was 30 per cent higher than for similar jobs in other areas. At the time, however, there was an unsettled labour situation which may have affected the tendering for concrete to a greater extent than for steel.

(c) In general, under normal competitive conditions it is found that reinforced concrete construction is less expensive than structural steelwork. In addition, in a recent 35-storey concrete building at 1550 Lake Shore Drive, Chicago the construction rate was one storey every three working days after construction had reached the 5th floor.

The AUTHOR stating his reason for giving a written reply comments :—

Mr. Hill introduced, at a meeting which assembled to consider and discuss lightweight fire protection, a carefully prepared statement containing a number of generalizations, and vague claims generally in support of concrete casings. These led to the suggestion that the cost ratios introduced by the Author, particularly in Figure 3 were incorrect. The Author was only handicapped by his inability to memorise at once a fairly complicated statement running to some 500 words. He felt at the meeting that he would do more justice to Mr. Hill by taking his statement in detail, than he would by replying verbally to a partially memorised version of it.

Mr. Hill felt that the Author had overstated his case. In fact the Author was at some pains not to do so. The building in each scheme generally complied with the requirements of the Model By-Laws. Scheme 1 epitomised the concrete cased steel frame designed in accordance with the recommendations of B.S.449 : 1948, with which Structural Engineers are more or less familiar depending on their particular calling. It is incidentally, the type of structure over which reinforced concrete has gained an ascendancy during the past decade.

Scheme 2 represented the opposite extreme in terms of the dead weight of fire protective casing.

Scheme 3 was conceived so as to take advantage of the best of both worlds and to reflect to what extent the advances in structural design recommended by B.S.449 : 1959 and which had any bearing on fire protection, would influence the cost of the structure.

The Author pointed out that the increased stresses had not been exploited in the design of Scheme 3 and he mentioned a number of incidental advantages, attendant upon lightweight fire protection alone, which had not been quantified in arriving at the final valuation of Scheme 3.

Mr. Hill will no doubt agree in view of these comments that the basic approach to the problem at any rate did not "overstate the case."

Mr. Hill asserts that the concrete casing increases the load carrying capacity of the member.

The paper states briefly how this was applied to column design, namely by stiffening the column about its weaker axis ; and after the publication of B.S.449 : 1959 by taking a proportion of the axial load on the column, as well.

Mr. Hill's generalization does not apply to the beams supporting the floors in the framed building under discussion. Mr. Gray made the point at the January meeting. A concrete casing to a beam has a structural value if, and only if, it imparts lateral restraint against compression flange buckling. Floor beams derive the restraint they need from the floor alone : the weight of the casings must be subtracted from the payload that the steel beam would otherwise carry. In other words the weight of concrete casings on beams is an extra burden on the steel frame and foundations. It would be a necessary burden if the concrete casing were cheaper than the plaster casting. It is not.

There is therefore no point in redesigning Scheme 1 to the recommendations of the new B.S.

However, Mr. Hill endeavoured to evaluate Scheme 1 on the basis of the Author's data and some information provided by Mr. C. S. Gray, and this requires

a comment because Mr. Hill did not get his facts quite right.

Mr. Gray had said in effect that the application of the new B.S.449 led to an economy of 15 per cent in the cost of concrete encased structures. "Structures" is the operative word, "concrete encased" is merely an adjectival phrase.

Accepting Mr. Hill's breakdown of 10 per cent due to the increased structural value of the concrete and 5 per cent due to higher stresses in the steel, and neglecting the latter, since economies due to higher stress put the comparison slightly out of focus, we find that if Scheme 1 were redesigned to the new B.S., the cost of the steelwork would be reduced by 10 per cent. The steelwork represents approximately one third of the superstructure cost in Figure 2 and therefore a saving of 10 per cent on the steel represents a saving of only 3⅓ per cent of the combined cost.

In other words the index figure for a fully concrete encased structure based on B.S.449 : 1959 would be 96·67, not 90 as Mr. Hill has led himself to believe. With this corrected value the index for Schemes 2 and 3 at 92·2 and 85·0 respectively appear to follow a logical sequence. Incidentally, if the above reasoning is applied to the data for Scheme 3—Figure 3, it would appear that the index figure for "Superstructure" would reduce from 85 to 83·6 owing to the increased stresses in the steelwork.

Very briefly, Mr. Hill's case was built up on, first, misquoting the Author; second, mistaking the value of concrete casings on floor beams, and third misunderstanding Mr. Gray on the subject of structural steelwork economy.

The Author would agree with Mr. Hill that when the concrete casing can be used for other purposes in addition to fire protection then it merits a fresh set of considerations before its value can be fully assessed.

For example, in each of the Schemes 1, 2 and 3, the external beams at every floor level were concrete cased because the casing served four entirely different purposes, as follows :—

(a). Fire protection
(b). Protection from weather
(c). Supporting external brickwork
(d). Partly decorative

As far as protection from corrosion is concerned this should be separated under at least two headings for comment; namely Industrial Building, and buildings such as hotels, flats, hospitals, offices and the like.

The intensity and nature of the corrosive attack inside industrial buildings often requires a specialised study and it is the Author's opinion that a concrete casing or a reinforced concrete structure will almost invariably represent the best line of defence, either because of the resistance of the concrete itself to some corrosive attacks or because it acts as a suitable background for whatever anti-corrosive treatment may be necessary in more severe conditions.

Buildings in the second category are all subject to the statutory requirements of the Thermal Insulation Act, and the provision of some form of central heating is becoming an accepted standard, with the result that the temperature of the whole carcase of the building is more stable and thus the risk of precipitation of moisture, the prelude to rusting, is reduced.

The Author had the opportunity of examining much of the steelwork over the auditorium of the Tivoli Theatre, Strand, during the demolition of that building. It dated from 1923 or thereabouts and was certainly more than 30 years old. Much of it was unpainted and it had been disguised from within by plaster relief work. There was no suggestion of rust anywhere.

"Dubbing out" steel beams and plastering them has been common practice particularly in houses and in the conversion of shops for half a century without particular anti-corrosive treatments and the Author knows of no recorded instance where this has led to trouble. He firmly believes he would have heard by now.

Preparing the comparison of costs of different building techniques on what is fundamentally the same structure is a precarious pastime, but it has the saving grace that any minor errors which might occur are probably applied throughout so that the comparison is not greatly invalidated.

Comparing the costs of fundamentally different structural media on the basis of general conditions is largely a waste of time.

As evidence of this, consider the Author's figures showing the comparative costs of reinforced concrete and plaster encased steel in the same buildings. Mr. Hill, although less explicit, asserts that some authorities have quoted the saving due to the use of reinforced concrete as high as 35 per cent compared with cased steelwork. Until such things as what types of building, when were they built, where were they built, what was the column layout, what was the span of beams, what was the floor loading, etc., are known it is impossible to get the two claims in a proper perspective. It is the Author's opinion that when all these factors are taken into account in one kind of building, structural steelwork is the logical choice, and in another reinforced concrete is better; there is no other general rule.

The Author's research, and subsequent practical experience has shown that vermiculite plaster casings contribute to substantial economies in the use of structural steelwork and a reduction of building costs, which hitherto have not been fully exploited.

Therein lay the Author's motive in bringing the matter to the attention of his profession.

In a written reply the Author would like to explain to Mr. Godfrey the reasons for concentrating on vermiculite gypsum plaster.

Most of the alternatives are scheduled in the Model By-laws and the London Building By-laws; details of them are more or less common knowledge among engineers.

The nine different forms of lightweight fire protection in the Bylaws do not appear greatly to have excited the attention of engineers or their architect colleagues, and it was the Author's intention to bring this apparently neglected subject more to the forefront.

The advantages of lightweight fire protection such as economy of steelwork, shorter construction time, etc., are available to any chosen system.

Vermiculite gypsum plaster, the newest fire protection medium does possess certain advantages over its rivals, for example :—

1. It is merely an extension of the plastering which is common in most multi-storey buildings.
2. It is lighter and cheaper than its contemporaries.
3. The fire resistance of concrete slabs is enhanced by vermiculite gypsum plasters on the ceilings. Therefore, with an identical material on the beams, there will be a continuous fire protective membrane minus the joints which must be present with any combination of other fire protective materials.

These were felt to be valid reasons, apart from any others, for making vermiculite gypsum plasters the basis of the paper.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 24th November, 1960, at 5 p.m. Lt.-Colonel G. W. Kirkland, M.B.E.(Mil.), M.I.Struct.E., M.I.C.E. (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

STUDENTS

ASLETT, Malcolm John, of London.
ATKINSON, Miss Jennifer, of Harrogate, Yorkshire.
BUCKLEY, David George, of Dewsbury, Yorkshire.
CLEMMETT, Gordon, of Bolton, Lancashire.
ECHIKWA, Vincent Enyinnaya, of London.
GHOSH, Ajit Kumar, B.Sc., of London.
GORTON, Stewart William, of Manchester.
HUME, Ian James, of Ilford, Essex.
HWANG, Liang Doh, of Singapore.
LOCKE, Joseph, of Bolton, Lancashire.
McKAY, John James, of London.
MOORES, Arthur Keith, of Bolton, Lancashire.
NELSON, Colin, of Bolton, Lancashire.
O'CONNOR, John Oliver, of Dublin, Eire.
OKAFOR, Chikwudozie Ebenezer, of London.
PARKER, George David, of Belfast.
SHEPHERD, Ian Robert, of Bury, Lancashire.
SHEPPARD, Brian Richard, B.Sc., of Richmond, Surrey.
SILCOCK, Miss Rosemary Winifred, of Lowestoft, Suffolk.
STAGG, Roger Ronald, of London.
STOCKS, Keith Bernard, of London.
STUDLEY, Royston David, of Bristol.
THAM SOON YOON, of Singapore.
TOMLINSON, John Lloyd, of Darlington, Co. Durham.
TUCKER, William Godfrey Lionel, of Swansea, South Wales.
YEO CHIEW HOON, of Singapore.
YOON YUEN WAH, of Kuala Lumpur, Malaya.

GRADUATES

ABU-SITTA, Salman Husein, B.Sc., of London.
ADIGUN, Herbert Folorunso, of London.
AHMAD, Mumtaz, B.Sc., of London.
ANDERSON, David Graham, A.M.I.C.E., of Cardiff, Glamorgan.
AU-YONG, Chin Koon, B.Sc., of Hong Kong.
AZAM, Khalid, B.Sc., of London.
BANDYOPADHYAY, Amarendra Mohan, B.E., of Manchester.
BASU, Tarun Kumar, B.E., of Bombay, India.
BEALE, Donald Edward, of Felbridge, Sussex.
BEDFORD, John Lewis, of Barkingside, Essex.
BERRISFORD, Carl James, of London.
BHATE, Manohar Dinkar, B.E., of Sangli (Maharashtra State), India.
BHUTTA, Fayyaz Ahmad, of London.
BICKERTON, Geoffrey Michael, of Harwood, Nr. Bolton, Lancashire.
BLAND, Peter Dudley, of Dewsbury, Yorkshire.
BROOKS, Donald, of Bury, Lancashire.
CARNIE, William Colin Greig, of London.
CARRICK, John Leslie, of London.
CHALMERS, William Kinnaird, of Garrowhill, By Baillieston, Lanarkshire.
CHAN CHIANG HENG, of Kuala Lumpur, Malaya.
CHANG, Tai Hon Philip, B.Sc., of Hong Kong.
CHARGE, John, of Manchester.
CHEOK YEOW KWANG, of Johore Bahru, Malaya.
CLARKE, Francis Gerard, of Radcliffe, Lancs.
COOK, Roy John Leonard, B.Sc., of Camberley, Surrey.
JOHNSTON, John Cooke, of Jamshedpur, India.
CRABTREE, Geoffrey, of London.
DALAL, Rusi K., B.E., of London.
DEDIARE, Tselogun, of London.
DELANEY, James Daniel Patrick, of London.
DE LORD, Bernard Pereira, of London.
DEWEY, William David, of Staines, Middlesex.
DOUGLAS, James Pollock Donaldson, of Corstorphine Edinburgh.
FOO FOOK FOON, of Brighton, Sussex.
GIBLENN, Alan Thomas, of Hoddesdon, Herts.
GLEAVE, Derek, of Warrington, Lancashire.
GODFREY, Terence John, of Calabar, Nigeria.
GOPAL, Mysore Krishniyengar, B.E., of Gauhati (Assam), India.
HALL, Robert Trevor, of Jersey City, New Jersey, U.S.A.
HALLETT, Rex, of Harrow Weald, Middlesex.
HARDING, Thomas, of Blackpool, Lancashire.
HAYES, Arthur, of Manchester.
HOPE, Frederick George, of Woking, Surrey.
HUNT, Cyril George, of Pontypridd, Glamorgan.
IDLE, Antony Herbert, B.Sc., of London.
IDOWU, Julius Olayinka, of London.
JACKSON, Trevor Hubert, of Binley, Coventry.
JACQUES, Trevor, of Chesterfield, Derbyshire.
JAMES, William, B.Sc.(Eng.), of Natal, South Africa.
KEECH, Barry, of Crawley, Sussex.
KINGHORNE, Roydel Patrick Aloysius, of Harlow, Essex.
LAKHANI, Vray Lal, B.Sc., of London.
McLOUGHLIN, Kevin Joseph, B.E., of London.
MADRASWALLA, Taher, B.E., of London.
MAH, Michael Guan Pang, of London.
MARSHALL, Kenneth Arthur, of Copthorne Bank, Nr. Crawley, Sussex.
MATHEWS, Brian John, of West Byfleet, Surrey.
MENZIES, John Barrie, B.Sc., of Boston, Lincolnshire.
MIELNIK, Janusz, B.Sc., of Birmingham.
MILLS, James, of Glasgow.
MISSELDINE, Derek Victor, of Harlow, Essex.
MOGHE, Nilkanth, Manohar, B.E., of Shivajinagar, Poona, India.
MOORE, Michael George, B.A., of Hyde, Cheshire.
MORGAN, Gerald John, A.M.I.Mun.E., of Wakefield, Yorkshire.
MORTON, Brian Anthony, of London.
NADKARNI, Arvind Putappa, of Bombay, India.
NEKOO, Rustom Keikhusroo, B.Sc.. of Nairobi, Kenya.
NORTH, David Greenwood, of Wilmslow, Cheshire.
PALMER, David Ernest, B.A., of Bromley, Kent.
PARDEY, Kenneth James, of Rayleigh, Essex.
PORTER, Alan George of South Croydon, Surrey.
PRESTON, Raymond Henry, of Teddington, Middlesex.
REDGROVE, Ronald Frederick, of London.
RODEN, Ian Anthony, of Stockport, Cheshire.
ROWE, John Albert, of Enfield, Middlesex.
SAMANT, Avinash Raghunath, B.E., of Domblivi, Maharastra State, India.
SANDERSON, John James, of Salisbury, Southern Rhodesia.

Scott, Douglas George, of Musselburgh, Midlothian, Scotland.
Setlur, Achyut Vencatranga, B.E., of Bombay, India.
Shah, Aravind Dhondichand, B.E., of Latur (Dist.: Osmanabad), India.
Sharma, Om Parkash, of Jangpura, New Delhi.
Sheehy, Morgan, B.E., of London.
Skinner, Donald Arthur James, of Backwell, Nr. Bristol, Somerset.
Smith, Anthony Lloyd, of Scunthorpe, Lincolnshire.
Snoad, John Edwin, of London.
Sutton, Derek John, of Solihull, Warwicks.
Tsui, Tack Kong, of Middlesbrough, Yorkshire.
Viswanathan, Tanjore Ramaswami, of Bhopal M.P., India.
Waldram, Alan Joseph, of Rothley, Leicester.
Ward, Gerald William, of Hounslow West, Middlesex.
Wallace, Reginald Rutherford, B.Sc., of London.
Wallis, Richard Allan, of Liverpool.
Wallwork, James Fowler, of Manchester.
Westal, Malcolm Charles, of London.
White, Keith Christopher, B.Sc., of Epsom, Surrey.
Whiting, Peter Roy, A.M.I.Mun.E., of Sutton Coldfield, Warwickshire.
Wilson, Robin Lee, B.Sc., A.M.I.C.E., of London.
Windross, Keith Stanley, of Middlesbrough, Yorkshire.
Wright, Ronald Edward, of Harlow, Essex.
Young, Dennis Richard, of Lichfield, Staffordshire.
Young, James, of Salfords, Nr. Redhill, Surrey.
Yuen Wai To, B.Sc., of Hong Kong.

Associate-Members

Nasmith, John Struan Irving, of Chichester, Sussex.
Newstead, William Ronald, of Middlesbrough, Yorkshire.
Wise, Peter Rodney, of Manchester.

Members

Chadda, Sita Ram of London.
Harris, Alan James, B.Sc., M.I.C.E., of London.
Piatkowski, Alojzy, M.Sc., M.I.C.E., of Lagos, Nigeria.
Saywood, Reginald Percival, of Bristol.
Sheppard, William Thomas Edwin, of Lower Kingswood, Surrey.

Transfers

Students to Graduates

Barton, Roger Ernest, of Hornchurch, Essex.
Boswell, Colin Joseph, of Birmingham.
Brett, David Edward, of Twickenham, Middlesex.
Fatehi, Taher Abdulhusein, B.Sc., of Manchester.
Fisher, Frederick James, of Aveley, Nr. Purfleet, Essex.
Fraser, John Scott, of Greenford, Middlesex.
Goodwin, Harry, of Stirling, Scotland.
Gregory, Henry Catchick, of London.
Halliwell, Garrod Yates, of Stockport, Cheshire.
Hellyer, David John, of New Malden, Surrey.
Luscombe, Glynn Michael William, of Bristol.
Maudlin, Ralph Henry, of London.
Parr, John, of Rainhill, Nr. Liverpool.
Taylor, Eric Henry Frank, of Partington, Cheshire.
West, William Harold, of Edinburgh.

Graduates to Associate-Members

Barczynski, Miroslaw, of London.
Burke, Terence, of Northwich, Cheshire.
Farnham, John Albert, of London.
Girardier, Edward Victor, of Timperley, Altrincham, Cheshire.
Gizejewski, Krystian, of London.
Howard, Douglas John of Sidcup, Kent.
Jones, Ian, of Culcheth, Nr. Warrington, Lancashire.
Khare, Nagesh Gundopant, B.E., of Poona, India.
Matthews, Douglas Thomas, of Bebington, Cheshire.
Menzies, James Charles, of Te Karaka, Via Gisborne, New Zealand.
Pardoe, Frederick James, B.Sc., of Birmingham.
Patel, Kantilal Varadhbhai, B.E., of Bristol.
Plummer, Ivor William, of Hemel Hempstead, Herts.
Puddephatt, George Ernest, B.Sc., of Flackwell Heath, Bucks.
Rees, Ronald Sterling Lloyd, A.R.I.B.A., of Guildford, Surrey.
Reynolds, David Edward Holden, of Scunthorpe, Lincs.
Sheffield, Peter, of Toronto, Canada.
Sillett, Donald Frank, of London.
Smith, Tony Clive, of New Normanton, Derby.
Sowerby, Paul Leon, of Manchester.
Stafford, Alex James, of Pilling, Nr. Preston, Lancashire.
Stapleton, Roger Louis, of Barnet, Herts.
Stephenson, Ralph, of Manchester.
Tynan, Derek Francis, of Crowle, Worcestershire.
Weston, John, of Pensby, Nr. Heswell, Cheshire.

Associate-Members to Members

Alchin, Frederick George, of Beckenham, Kent.
Banerjee, Bhola Nath, B.E., of Calcutta, India.
Bond, Leslie Kenneth, of Cheam, Surrey.
Brownlow, Harold Bertram Stanislaus, of Holyrood, Newfoundland.
Clarke, Godfrey Baynes, of Sanderstead, Surrey.
Dale, Hilary Michael, A.M.I.C.E., of Sanderstead, Surrey.
De Courcy, John Wray, B.E., M.E., M.I.C.E.I., of Dublin, Ireland.
Dowe, Horace William, of Leeds.
Evans, Robert Edgar, of London.
Hill, Frederick Alfred, of Maidenhead, Berks.
Jones, Professor Royston, M.Sc.Tech., Ph.D., M.I.Mun.E., A.M.I.C.E., of Accra, Ghana.
Koeppler, Hans, of Johannesburg, South Africa.
Peacock, John Desmond, B.Sc., A.M.I.C.E., of Sunbury-on-Thames, Middlesex.
Sefton, Wolf, B.Sc.(Eng.), M.I.C.E., of Toronto, Ontario, Canada.
Smith, James Maclagan, M.I.C.E., A.M.I.Mun.E., of Preston, Lancashire.
Stevens, John Thomas, A.M.I.Mech.E., of Singapore.
Tandler, Herbert Kurt, B.Sc., of Johannesburg, South Africa.
Tighe, Thomas Raymond, of Redcar, Yorkshire.
Wall, Victor Grayson, of Liverpool.

OBITUARY

The Council regrets to announce the deaths of William Robert Francis Connell, Stanley Kirkham Shufflebotham, Jack Leslie Wheeler (Members); Captain Harry George Hewitt, David John Morris, Rubert Henry Squire (Retired Members); William Henry Appleyard, George Beckett, Harold Entwistle, Leonard Charles Free, Ram Lal Loomba, John Salmond Murray, Janakray Rambhai Patel, Samuel Young-Wai (Associate-Members); Phillip Richard Dunkley (Retired Associate-Member); Anthony Roy Compton, John Lomax Yates (Graduates).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, 12th January, 1961

Ordinary Meeting at 6 p.m., when Mr. A. Short, M.Sc., M.I.Struct.E., and Mr. W. Kinniburgh, F.R.I.C., will give a paper on "The Structural Use of Aerated Concrete."

Thursday, 26th January, 1961

Ordinary General Meeting for the election of members, 5.55 p.m. This meeting, which is open only to corporate members of the Institution, will be followed by an Ordinary Meeting at 6 p.m., when Mr. P. B. Edwards, A.M.I.Struct.E., and Mr. R. B. Rigg will give a paper on "The Design and Construction of Extension to British European Airways Engineering Base at London Airport."

Thursday, 9th February, 1961

Ordinary Meeting at 6 p.m., when Mr. J. B. Dwight, M.A., M.Sc., A.M.I.Mech.E., will give a paper on "Aluminium Strut Design."

Thursday, 23rd February, 1961

Ordinary General Meeting for the election of members, 5 p.m. The paper which was to have been given on this date has been postponed until the 9th March.

Thursday 9th March, 1961

Ordinary Meeting at 6 p.m., when Professor S. R. Sparkes M.Sc., Ph.D., M.I.Struct.E., M.I.C.E. (Member of Council) and Dr. J. C. Chapman will give a paper on "Model Methods, with particular reference to Three Recent Applications in the Field of Steel, Composite and Concrete Construction."

Thursday, 23rd March, 1961

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. E. M. Lewis will give a paper on "Rubber Tyred Overhead Cranes—Their Influence on the Design of Industrial Buildings."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS, JANUARY, 1961

The Examinations of the Institution will next be held in the United Kingdom and overseas on the 10th and 11th January, 1961 (Graduateship) and the 12th and 13th January, 1961 (Associate-Membership).

PRIZES—JULY 1960 EXAMINATIONS

The Council have awarded the following prizes in respect of the examinations held in July, 1960 ;-

ANDREWS PRIZE.—For the candidate who obtains the highest aggregate of marks in the Associate-Membership Examination, passing in all subjects.
D. J. HOWARD of Sidcup, Kent.

HUSBAND PRIZE.—For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the paper "Structural Engineering Design and Drawing."
G. GARNER of Wolverhampton.

WALLACE PREMIUM (SENIOR).—For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the paper "Theory of Structures (Advanced)."
D. J. HOWARD of Sidcup.

A. E. WYNN PRIZE.—For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the Reinforced Concrete Section 'C' of the paper "Structural Engineering Design and Drawing."
G. GARNER of Wolverhampton.
R. S. CARCAS of Bath.

GRAHAM WOOD PRIZE.—For the candidate who obtains the highest marks in the Steel section 'B' of the paper "Structural Engineering Design and Drawing" in the Associate-Membership Examination.
J. R. TREZIES of London.

WALLACE PREMIUM (JUNIOR).—For the most successful candidate in the Graduateship Examination, passing in all subjects.
W. L. JONES of Cwmbran, Mon.

EXAMINATION PRIZES—Revised Conditions

The conditions of the awards of the A. E. Wynn and Graham Wood Prizes have been revised. The new conditions are as follows :—

A. E. WYNN PRIZE.—For the candidate of those submitting a reinforced concrete design who, having passed or been exempted from Section 'A,' obtains the highest marks in Section 'B' of the Associate-Membership Examination.

GRAHAM WOOD PRIZE.—For the candidate of those submitting a structural steelwork design who, having passed or been exempted from Section 'A,' obtains the highest marks in Section 'B' of the Associate-Membership Examination.

AUCKLAND (NEW ZEALAND) SECTION

The Council have sanctioned the formation of an Auckland (New Zealand) Section of the Institution with headquarters in Auckland. The Honorary Officers and Committee members are as follows :—

Chairman : Mr. William H. S. Nixon (Member).
Honorary Secretary : Mr. A. Donald (Associate Member).
Committee Members : Messrs. Noel Gedye A. Slater, D. MacGregor.

Communications should be addressed to Mr. A. Donald, B.Sc.(Hons.), A.M.I.Struct.E., 122 Matipo Road, Te Atatu, Auckland, New Zealand.

REPRESENTATION

The following Institution representatives have been appointed :—

British Iron and Steel Research Association, Advisory Panel, Trials of Painting Schemes for Sprayed Metal Coatings :
Mr. G. B. GODFREY.

BSI Glossary of General Building Terms Committee—BSI Schedule of Weights of Building Materials Technical Committee :
Mr. D. T. WILLIAMS.

Lanchester College of Technology, Civil Engineering Advisory Committee :
Mr. H. W. THROWER.

REPRESENTATION OVERSEAS

The Council have appointed Mr. J. R. M. MacIntyre (Associate-Member), to be the Institution's Representative in Singapore in place of Mr. W. N. Cursiter, who has taken up a new appointment in Nigeria.

SESSIONAL PROGRAMME

The Literature Committee have under consideration the selection of papers for inclusion in the Sessional Programme for 1961-2. Members who may wish to offer papers during the coming Session are invited to communicate with the Secretary.

ARTICLES FOR PUBLICATION

The Literature Committee would be glad to consider short articles of a descriptive character for publication in the Journal. The object of such articles would be to describe the difficulties encountered during construction and the ways in which they were overcome; and to set out details of structures successfully erected. Articles should be accompanied by good illustrations.

ANNUAL DINNER, 1961

The Annual Dinner of the Institution will be held at the Dorchester Hotel on Friday, 5th May, 1961.

ADDITIONS TO THE LIBRARY

APCAR, H. V. *Structural Frameworks for Single-Storey Factory Buildings*, London, 1960.

CEMENT & CONCRETE ASSOCIATION. *Design and Construction of Open Air Swimming Pools*, London, 1960.

CHARLTON, T. M. *Energy Principles in Applied Statics*, London, 1959. Presented by Mr. S. Woolf.

GODFREY, G. B. *The Use of Welding in Steel Building Structures*, London, 1960.

GRAHAM, L. A. *Ingenious Mathematical Problems and Methods*, New York, 1959.

GUYON, Y. *Béton Précontraint.—Etude Théorique et Experimentale, Tome II Constructions Hyperstatiques*, Paris, 1958; *Prestressed Concrete, Vol. II Statically Indeterminate Structures*, London, 1960. Presented by Dr. P. W. Abelcs.

Institution of Engineers Ceylon Transactions, 1959, Colombo, 1959.

KINNUNEN, S. and NYLANDER, H. *Punching of Concrete Slabs without Shear Reinforcement*, Stockholm, 1960.

MAINSTONE, R. J. *Studies in Composite Construction, Part III, Tests on the New Government Offices Whitehall Gardens*, London, 1960.

MURPHY, G. M. *Ordinary Differential Equations and their Solutions*, New York & London, 1960.

NORRIS, C. H. and WILBUR, J. B. *Elementary Structural Analysis, 2nd Edition*, New York & London, 1960.

RAMASWAMY, G. S. and OTHERS. *A Shell of Double Curvature for Roofs and Floors*, India, Roorkie.

REIMBERT, M. & A. *Calcul rapide des poutres continues par la méthode de M. Caquot*, Paris, 1960. Presented by Mr. Leslie Turner.

RITCHIE, C. C. *Studies on Bridge-Deck Systems, II Bending Moments in a Plate-and-Girder System*, London, 1960.

ROCARD, Y. *General Dynamics of Vibrations*, London, 1960.

TIMBER DEVELOPMENT ASSOCIATION. *The Use of Timber in Dock and Harbour Engineering*, London, 1960.

VÖLTER, O. *Grout, Grouting Techniques and the Construction of the Prestressing Tendon*, London, C. & C.A. Library Translation No. 90.

WEBB, H. A. & ASHWELL, D. G. *A Mathematical Tool-Kit for Engineers*, London, 1960.

WENTWORTH-SHEILDS, F. E., GRAY, W. S. and EVANS, H. W. *Reinforced Concrete Piling and Piled Structures, 2nd Edition*, London, 1960.

Branch Notices

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Tuesday, 17th January, 1961

Mr. J. H. A. Crockett, B.Sc., A.M.I.Struct.E., A.M.I.C.E., on "Vibration Problems in Relation to Foundation Design and Construction."

Friday, 3rd February, 1961

Annual Dinner Dance in the Derby Suite, Midland Hotel, Manchester.

Wednesday, 15th February, 1961

Joint Meeting with the Institute of Welding. "The Influence of Welding on Structural Design," by Mr. W. Bates, M.I.Struct.E. (Past Chairman—Lancashire and Cheshire Branch).

Tuesday, 14th March, 1961

"Railway Bridge Work in the Liverpool/Manchester Area," by Mr. F. Turton, M.I.Struct.E.

Unless otherwise stated, meetings will be held at the College of Science and Technology, Manchester, commencing at 6.30 p.m. Light refreshments will be available from 5.45 p.m.

MERSEYSIDE PANEL

The following meetings have been arranged :—

Monday, 23rd January, 1961

"Prestressed Concrete and C.P. 115 by Mr. A. J. Harris, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., M.Cons.E.

At Liverpool University in the New Civil Engineering Building, Brownlow Hill.

Tuesday, 21st February, 1961

Junior Members Evening.

At the College of Building, Clarence Street, Liverpool.

Monday, 20th March, 1961

"Behaviour of Tubular Structures in Compression—Static and Dynamic" by Professor Sir Alfred Pugsley, O.B.E., D.Sc.(Eng.), F.R.S., M.I.Struct.E., M.I.C.E., F.R.Ae.S. (Past President).

At the Liverpool University in the new Civil Engineering Building, Brownlow Hill.

All meetings will commence at 6.30 p.m., and will be preceded by light refreshments from 5.30 p.m.

Joint Hon. Secretaries: Wm. S. Watts, M.I.Struct.E., A.M.I.C.E., 11, Newchurch Lane, Culcheth, Nr. Warrington, Lancs., and M. D. Woods, A.M.I.Struct.E., 8, Dennison Road, Cheadle Hulme, Cheshire.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Friday, 27th January, 1961

"The Structural Engineer in the Field of Atomic Energy" by T. C. Waters, M.I.Struct.E. (Member of Council), Joint meeting with the Institute of Welding, Birmingham Branch.

Friday, 24th February, 1961

"Computers and Structural Design—The Present Position and Future Trends" by Mr. R. E. Rowe, M.A., M.I.C.E.

Tuesday, 7th March, 1961

"Mining Subsidence, Design Problems and Remedial Measures" by Mr. K. Wardell, F.R.I.C.S., M.I.M.E., F.G.S.

At the East Midlands Electricity Showrooms, Irongate, Derby, at 6.45 p.m. Tea will be served at 5.45 p.m.

Friday, 24th March, 1961

"Models for Design and Research in Reinforced Concrete" by Dr. Geoffrey Brock, A.M.I.C.E.

Unless otherwise stated, all meetings will be held at the Byng Kendrick Suite, College of Advanced Technology, Gosta Green, Birmingham, 6, at 6.30 p.m. Tea will be served from 5.45 p.m.

Hon. Secretary: S. M. Cooper, M.I.Struct.E., 'Applegarth,' Hyperion Road, Stourton, Nr. Stourbridge, Worcestershire.

GRADUATES' AND STUDENTS' SECTION

Friday, 6th January, 1961

"Precast Concrete Frame Construction," by Mr. Bryan E. Griffin, A.M.I.Struct.E.

Friday, 20th January, 1961

Fourth Annual Buffet Dance.
At the Station Hotel, Dudley.

Friday, 3rd February, 1961

"Structural Engineering in the Construction of the Inner Ring Road," by Mr. S. G. Piggot, Principal Engineering Assistant in Birmingham Public Works Structural Department.

Friday, 3rd March, 1961

"Report of a Visit to Brazil, Venezuela and Mexico," by Dr. D. D. Matthews, M.I.Struct.E., M.I.C.E. (Vice-President).

Unless otherwise stated, all meetings will be held at the Engineering Centre, Stephenson Place, Birmingham, and will start at 6.30 p.m. Tea will be served from 6 p.m.
Hon. Secretary: H. T. Dodd, Shepherd's Cottage, Grove Lane, Wishaw, Sutton Coldfield, Warwickshire.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, 3rd January, 1961

At Middlesbrough. Students' and Graduates' Meeting. Address by Mr. O. A. Kerensky, B.Sc., M.I.Struct.E., M.I.C.E., "Design of Welded Plate Girders."

Wednesday, 4th January, 1961

At Newcastle. Joint Meeting with the Northern Architectural Association. "Value for Money in Building," by Mr. E. S. Benson, M.B.E., M.A., F.R.I.B.A.

Wednesday, 1st February, 1961

At Newcastle. Students' and Graduates' Meeting. Address by Mr. W. E. J. Budgen, B.Sc., M.I.Struct.E., M.I.C.E.

Tuesday, 7th February, 1961

At Middlesbrough. Joint Meeting with the Northern Counties Association of the Institution of Civil Engineers. 5.45 p.m. for 6.15 p.m. "The Channel Crossing—Bridge or Tunnel" by Mr. J. F. Pain, M.C., B.Sc.(Eng.), M.I.C.E.

Wednesday, 1st March, 1961

At Newcastle. "The Design, Fabrication and Erection of Welded Girders for the South Durham Steel & Iron Co. Ltd., Greatham," by Mr. W. G. Gentry, M.I.Struct.E., A.M.I.C.E.

Tuesday, 7th March, 1961

At Middlesbrough. The above paper will be repeated.

Unless otherwise stated, meetings in Newcastle will be held at the Neville Hall and those in Middlesbrough at the Cleveland Scientific and Technical Institution, will commence at 6.30 p.m., preceded by buffet tea at 6 p.m.
Hon. Secretary: P. D. Newton, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., Messrs. S. Cussons & Partners, 112, Borough Road, Middlesbrough.

NORTHERN IRELAND BRANCH

The following Meetings have been arranged :—

Tuesday, 3rd January, 1961

Mr. C. H. Bunclark, M.I.Struct.E., will give a lecture and show a film on Piling.

Tuesday, 7th February, 1961

"Concrete Bridge Design," by Mr. L. Clements, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E.

Tuesday, 7th March, 1961

Annual Dinner.

Meetings will be held in the Civil Engineering Department, David Keir Building, Queen's University, Belfast, at 6.30 p.m., unless otherwise notified. Tea will be served from 5.45 p.m. to 6.30 p.m.
Hon. Secretary: L. Clements, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., 3, Kingswood Park, Cherryvalley, Belfast, 5.

SCOTTISH BRANCH

The following meetings have been arranged :—

Tuesday, 24th January, 1961

"The Provisions of the Revised British Standard for the Use of Structural Steel in Building—B.S. 449/1959," by Mr. Walter C. Andrews, O.B.E., M.I.Struct.E. M.I.C.E. (Past President).

Wednesday 25th January, 1961

Meeting at Edinburgh.—*See* Edinburgh Section.

Friday, 17th February, 1961

Joint Meeting with the Glasgow and West of Scotland Association of the Institution of Civil Engineers. "The Development of Nuclear Power in the United Kingdom," by Mr. T. C. Waters, M.I.Struct.E., (Member of Council, Past Chairman, Lancashire and Cheshire Branch).

Tuesday, 21st March, 1961

"The Design and Construction of the Pannier Market at Plymouth," by Mr. A. P. Mason, B.Sc.(Hons.), M.I.Struct.E., M.I.C.E. (Past Chairman, Midland Counties Branch).

Unless otherwise stated, meetings will be held at The Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, commencing at 7 p.m.

EDINBURGH SECTION

Wednesday, 25th January, 1961

"The Provisions of the Revised British Standard for the Use of Structural Steel in Building—B.S. 449/1959," by Mr. Walter C. Andrews, O.B.E., M.I.Struct.E., M.I.C.E. (Past President).

At the Heriot-Watt College, Edinburgh, 6.30 p.m.
Hon. Secretary: W. Shearer Smith, M.I.Struct.E., A.M.I.C.E., c/o The Royal College of Science and Technology, George Street, Glasgow, C.1.

SOUTHERN BRANCH

The following meetings have been arranged :—

Friday, 27th January, 1961

Mr. E. W. H. Gifford, B.Sc., M.I.Struct.E., M.I.C.E., on "Precast Frames."

Friday, 10th March, 1961

Mr. H. Tottenham on "Timber Structures."
Hon. Secretary: A. P. K. Tate, B.Sc., A.M.I.Struct.E., Department of Civil Engineering, The University, Southampton.

SOUTH-WESTERN COUNTIES SECTION

The following meetings have been arranged :—

Friday, 27th January, 1961

Paper to be read by Mr. H. J. Scoles, L.R.I.B.A., M.I.Struct.E., Member of the Section Committee.

At the Victoria Hotel, Torquay. Tea at 5.30 p.m. for 6 p.m.

Friday, 24th February, 1961

Joint Meeting with the Institution of Civil Engineers. "The Choice of Protective Schemes for Structural Steelwork," by Mr. F. Fancutt (Assistant Director of Research B.T.C.) and J. C. Hudson (Head of Corrosion Section, British Iron and Steel Research Association) at the Duke of Cornwall Hotel, Plymouth. Tea at 5.30 p.m. followed by meeting at 6 p.m.

Friday, 24th March, 1961

Joint Meeting with the Cement and Concrete Association. "Mix Design and Quality Control of Concrete," by Mr. R. I. T. Williams, B.Sc., A.M.I.C.E., A.M.Inst.H.E., at the Duke of Cornwall Hotel, Plymouth, at 6 p.m., preceded by tea at 5.30 p.m.
Hon. Secretary: C. J. Woodrow, J.P., "Elstow," Hartley Park Villas, Mannamead, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged:

Wednesday, 1st February, 1961

Joint Meeting with the South Wales Institute of Architects, Western Branch. "The Planning and Design of the Central Market, Swansea," by Mr. Howel Menders, L.R.I.B.A., and Mr. J. M. Burke, M.A., A.M.I.Struct.E., at Swansea.

Thursday, 2nd February, 1961

Joint Meeting with the Institution of Civil Engineers and the Institution of Municipal Engineers. "Multi-Storey Car Parks" by Mr. E. C. Roberts, M.Eng., M.I.C.E., M.I.Mun.E., at Cardiff.

Tuesday, 21st February, 1961

"Economics of Framed Structures" by Mr. L. R. Creasy, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., at Cardiff.

Friday, 3rd March, 1961

Joint Meeting with the Reinforced Concrete Association. "Lift Slab Construction," by Mr. F. R. Benson, B.Sc.(Eng.), A.M.I.C.E., at Cardiff.

Friday, 17th March, 1961

Annual Dinner.
At the Langland Bay Hotel, Swansea.

Wednesday, 22nd March, 1961

Students' papers and films.
At Swansea.

Meetings at Cardiff will be held at the South Wales Institute of Engineers, Park Place, and will commence at 6.30 p.m.

Meetings at Swansea will be held at the Mackworth Hotel, High Street, and will commence at 6.30 p.m.
Hon. Secretary: W. D. Hollyman, A.M.I.Struct.E., 41, Greenfield Avenue, Dinas Powis, Glamorgan.

WESTERN COUNTIES BRANCH

The following meetings have been arranged:—

Friday, 6th January, 1961

Joint Meeting with the Reinforced Concrete Association. "Lift-Slab Design and Construction" by Mr. F. R. Benson, B.Sc.(Eng.), A.M.I.C.E.

Friday, 3rd February, 1961

Joint Meeting with the Bristol and Somerset Society of Architects. The meeting will be held as a Forum, the Panel answering questions submitted by members.
Panel—
Architect: Mr. M. J. Hislop, A.R.I.B.A., F.R.S.A.
Quantity Surveyor: Mr. R. H. Mildred, F.R.I.C.S., F.I.Arb.
Contractor: Mr. R. C. Lorraine, B.Sc.(Eng.).
Engineer: Mr. Lewis E. Kent, B.Sc., M.I.Struct.E., M.I.C.E., (Past President).
Question Master: The Chairman, Clifford E. Saunders, M.I.Struct.E.

Tuesday, 14th February, 1961

Annual Dinner and Dance.
To be held at the Ashton Court Country Club, Failand, Bristol, from 7.30 p.m. to 1.0 p.m.

Friday, 3rd March, 1961

Joint Meeting with the Institution of Civil Engineers (South Western Association). "Berkeley Power Station—Some Special Factors Influencing Design and Construction," by Mr. A. L. Brake, A.M.I.Struct.E.

All meetings will be held in the Small Lecture Theatre of the University Engineering Laboratories, University Walk, Bristol, 8, at 6 p.m., preceded by tea at 5.30 p.m.
Hon. Secretary: A. C. Hughes, M.Eng., A.M.I.Struct.E. A.M.I.C.E., 21, Great Brockeridge, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged:—

Wednesday, 18th January, 1961

At Leeds. "Research for the Concrete Industry" by Dr. A. R. Collins, M.B.E., M.I.Struct.E., M.I.C.E. (Member of Council).

Wednesday, 1st February, 1961

Details to be announced.

Wednesday, 15th February, 1961

At Leeds. "Bridge Construction" by Mr. E. R. Bowers, Works Superintendent, Dorman Long Ltd.

Wednesday, 1st March, 1961

At Hull. Joint Meeting with the Hull and East Riding Branch of the Institution of Civil Engineers. "The Design and Construction of the New Transit Sheds at No. 12 Quay, King George Dock, Hull" by Mr. R. A. Fisher, A.M.I.C.E.
In the Lecture Theatre, Electricity Building, Ferensway, at 6.15 p.m.

Wednesday, 15th March, 1961

At Leeds. "Water Excluding Structures" by Mr. L. R. Creasy, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., and Mr. L. Scott White, O.B.E., M.I.Struct.E., M.I.C.E. (Past President).

Tuesday, 21st March, 1961

At the Blue Bell Hotel, Scunthorpe, 6.15 p.m. Joint Meeting with the East Midlands Association of the Institution of Civil Engineers. "Civil Engineering Problems in Nuclear Power Station Construction," by Mr. A. L. Brake, A.M.I.Struct.E.

Friday, 24th March, 1961

At Leeds. Annual Dinner and Dance. The Griffin Hotel, 7 p.m.

Meetings at Leeds will be held at the Metropole Hotel, King Street, unless otherwise stated, and meetings at Sheffield at the Royal Victoria Hotel. Meetings will commence at 6.30 p.m. preceded by buffet tea at 6.15 p.m.
Hon. Secretary: W. B. Stock, A.M.I.Struct.E., 34, Hobart Road, Dewsbury, Yorks.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary: E. B. Kretzchmar, A.M.I.Struct.E., P.O. Box 3306, Johannesburg, South Africa.
Natal Section Hon. Secretary: J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P.O. Box 932, Durban.
Cape Section Hon. Secretary: R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

EAST AFRICAN SECTION

Hon. Secretary: K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

SINGAPORE AND FEDERATION OF MALAYA SECTION

Hon. Secretary: J. R. M. MacIntyre, A.M.I.Struct.E., c/o Redpath, Brown & Co. Ltd., P.O. Box 648, Singapore.

NIGERIAN SECTION

Hon. Secretary: A. Brimer, A.M.I.Struct.E., Brimer, Andrews & Nachshen, Private Mail Bag, 2295, Lagos, Nigeria.